U.S. DEPARTMENT OF COMMERCE
National Technical Information Service
PB80-127087

Water Reuse-From Research to Application Proceedings of Water Reuse Symposium Held at Washington, DC on March 25-30, 1979. Volume 2

American Water Works Association Research Foundation, Denver, CO

Prepared for

National Science Foundation, Washington, DC

Mar 79

PB80-127087

NSF/RA 790225

PROCEEDINGS— VOL. 2

March 25-30, 1979
Washington, DC

WATER REUSE SYMPOSIUM

Theme:
"Water Reuse—From Research to Application"

Co-sponsored by:

American Water V Foundation
Office of Water Re .S. Dept. of the Interior)
U.S. Army Medica nt Command
U.S. Environmenta
National Science Foundation
Water Pollution Control Federation

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

ASRA INFORMATION RESOURCES
NATIONAL SCIENCE FOUNDATION

CONTROL NUMBER
3. COL. CO.
LL DOCUMENT PROCESSING
BATCH NUMBER

WATER REUSE
SYMPOSIUM

EXECUTIVE COMMITTEE

Richard D. Heaton
posium Organizer & Administrator
VA Research Foundation
; West Quincy Avenue
ver, Colorado 80235
) 794-7711

Villiam J. Cooper
Army Medical Bioengineering
D Laboratory
ng 568, Fort Detrick
rick, Maryland 21701
663-7207

Walter P. Lambert
ED R&D
)-OPA)
etrick
ick, Maryland 21701
63-7301

nes V. Basilico
Community Sources Staff
Management Division
f Research and Development
vironmental Protection Agency
' Street, S.W.
gton, D.C. 20460
(202) 426-3974

Mr. Ken Suter
Committee Chairman
OWRT—Tech Transfer
U.S. Department of the Interior
18th and "C" Street, N.W.
Washington, D.C. 20240
(202) 343-6883

Mr. Robert Madancy
Chief, Engineering Systems Division
OWRT—U.S.D.I.
18th and "C" Street, N.W.
Washington, D.C. 20240
(202) 343-6481

Dr. Edward H. Bryan
Advanced Environmental Research
and Technology
National Science Foundation
1800 "G" Street, N.W.
Washington, D.C. 20550
(202) 632-4356

Dr. K. Daniel Linstedt
WPCF—Reuse Committee Chairman
Department of Civil, Environmental and
Architectural Engineering
University of Colorado
Boulder, Colorado 80309
(303) 492-7007

This 3-volume set of Water Reuse Symposium Proceedings is available for $15.00 from:

AWWA Research Foundation
6666 West Quincy Avenue
Denver, Colorado 80235

a

REPORT DOCUMENTATION PAGE

1. REPORT NO.
NSF/RA-790225

2.

3. Recipient's Accession No.
PB80-127087

4. Title and Subtitle
Water Reuse, From Research to Application, Proceedings of the Water Reuse Symposium (Washington, D.C., March 25-30, 1979) Vol. 2

5. Report Date
March 1979

6.

7. Author(s)

8. Performing Organization Rept. No.

9. Performing Organization Name and Address
American Water Works Association (AWWA) Research Foundation
6666 West Quincy Avenue
Denver, Colorado 80235

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C)
(G) PFR7821313

12. Sponsoring Organization Name and Address
Engineering and Applied Science (EAS)
National Science Foundation
1800 G Street, N.W.
Washington, D.C. 20550

13. Type of Report & Period Covered
Proceedings

14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

The Symposium was devoted to the renovation and reuse of wastewaters from municipal, industrial, and agricultural sources. Unique to the Conference was the use of poster presentation for enhancing author-participant communication. The proceedings demonstrate that significant global interest exists in wastewater reuse, that recycling concepts are being applied to a broad range of functions, that application of water reuse may be highly situation-dependent and that reuse applications are important solutions to conservation of water. Symposium objectives include provision of program overviews; presentations of research results and practical experiences; provision of a forum for discussing the problems of reuse implementation and encouragement of information exchange and technology transfer. Volume 2 focuses on international developments, institutional factors, and treatment R & D operations.

17. Document Analysis a. Descriptors

Water
Waste water reuse
Water reclamation
Research and development
Conservation
Industries
Proceedings
Meetings
Economic analysis
Law (jurisprudence)

b. Identifiers/Open-Ended Terms

Water reuse

c. COSATI Field/Group

18. Availability Statement
NTIS Also available as a 3-volume set from the AWWA Research Foundation

19. Security Class (This Report)

20. Security Class (This Page)

21. No. of Pages
860

22. Price

(See ANSI–Z39.18) See Instructions on Reverse

OPTIONAL FORM 272 (4–77)
(Formerly NTIS–35)
Department of Commerce

PROCEEDINGS - VOL. 2 of the

WATER REUSE SYMPOSIUM

THEME:

"WATER REUSE -- FROM RESEARCH
TO APPLICATION"

HELD:

MARCH 25-30, 1979

WASHINGTON, D.C.

Any opinions, findings, conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

FORWARD

Wastewater reuse is one of several conceptual approaches to water conservation. It is based upon the realization that not all water users need water of the same quality. The idea of having a given quantity of water serve several different or repetitive functions before reintroduction to the environment is not new, but the rate at which it is increasing is.

Implementation of water reuse depends upon successfully satisfying constraints derived from considerations for environmental and human health effects, economics, process water quality, legal factors, and user acceptability.

The Symposium was the first week-long meeting devoted entirely to the renovation and reuse of wastewaters from municipal, industrial and agricultural sources. Unique to the Conference was the use of poster presentations for enhancing author-participant communication. Where quality photos were available, they are included at the end of several written manuscripts presented as posters.

These three volumes of Proceedings demonstrate that significant global interest exists in wastewater reuse; that recycling concepts are being applied to a broad range of functions; that application of water reuse may be highly situation-dependent; and that reuse applications are important solutions to conservation of water.

The objectives of the Symposium were to:

1. Provide program overviews
2. Present research results and practical experiences
3. Provide a forum for discussing the problems of reuse implementation
4. Encourage information exchange and the transfer of technology

These objectives were addressed and met by the 155 Symposium papers.

No attempt has been made to edit, reformat, or alter the material provided except for printing production requirements or where obvious errors or discrepancies have been detected. Any statements or views here presented are totally those of the speakers and are neither condoned nor rejected by the Symposium co-sponsors.

TABLE OF CONTENTS

	Volume	Page
Organizing Committee & Proceedings Availability	Inside Cover	
Cover Page	1,2,3	i
Forward	1,2,3	ii
Table of Contents	1,2,3	iii
Author Index	1,2,3	xi
Sponsor Information Centers	1	xviii
WATER REUSE POLICY, IMPLEMENTATION, PROGRAMS AND POTENTIAL	1	1
Theme Address - Water Reuse from Research to Application	1	3
Water Reuse in the Department of Defense	1	6
Wastewater Reuse Program at the U.S. Army Medical Bio-Engineering Research & Development Laboratory	1	10
Keynote Address: The President's National Water Policy in Relation to Water Reuse	1	22
OWRT's Water Reuse R&D Program	1	28
Water Reuse: It is Time for Implementation	1	36
The EPA's Research Program for Water Reuse	1	41
Influence of the Clean Water Act on Wastewater Reuse Planning	1	48
Applied Science and Its Role in Addressing Problems Relating to Renovation and Reuse of Water	1	55
The National Science Foundation's Role in Support of Research Relating to Reclamation and Reuse of Wastewater	1	59
Water Reclamation Efforts in California	1	69
The Potential for Wastewater Reuse in the United States	1	85
WATER SUPPLY AUGMENTATION	1	103
Demonstration of Potable Water Reuse Technology -- the Denver Project	1	105
Groundwater Recharge at Water Factory 21	1	139
Wastewater in Drinking Water Supplies	1	167
Santa Clara Valley Water District's Water Reclamation Facility at Palo Alto	1	179
Siting of Water Intakes Downstream from Municipal Wastewater Facilities	1	192
Observations on the Start-up of the Water Reclamation Recharge Project in Nassau County, New York	1	199
Direct Recharge for El Paso, Texas	1	212

TABLE OF CONTENTS CONTINUED

	Volume	Page
Purecycle Corporation -- A Domestic Wastewater Recycling System	1	239
Treatment of Municipal Wastewater Effluent For Potable Water Reuse	1	246
Toward the Goal of Direct Water Reuse -- What Should we do Next?	1	260
INDUSTRIAL RECYCLING AND REUSE	1	269
The Contra Costa Reclaimed Water Project-- An Update	1	271
AWT Evaluation in Two Selected Industries	1	283
Total Water Reuse -- Can Industry Afford It?	1	295
Industrial Reuse of Wastewater: Cost Analysis and Pricing Strategies	1	301
Effects of EPA Guidelines on Industrial Water Reuse	1	323
Reuse of Reclaimed Food Processing Wastewater	1	336
Reuse of Municipal Wastewater as Makeup to Circulating Cooling Systems	1	371
Water Management Plan for a Midwestern Utility	1	398
Total Wastewater Reclamation and Reuse at McClellan Air Force Base, California	1	405
Water Reclamation and Reuse in the Power, Petrochemical Processing and Mining Industries	1	421
REDUX Water Recycling System	1	433
Industrial Wastewater Recycle/Reuse Technology	1	448
Recycling Systems for Mechanical Platers	1	462
Amarillo's Water Supply = 70% New + 30% Reused	1	467
Military Field Sanitation and Wastewater Reuse	1	474
Power Plant Condenser Cooling Water As a Water Supply Source	1	496
Reuse of Wastewater Containing Firefighting Agent	1	506
Coal Gasification Wastewater Reuse	1	522
PLANNING FOR REUSE	1	544
Regional Water Reuse in Southern California	1	546
Guidelines for Implementing a Water-Reuse Program	1	565

TABLE OF CONTENTS CONTINUED

	Volume	Page
Wastewater Reclamation For the San Francisco Bay Region	1	585
Wastewater Reuse Alternatives at South Lake Tahoe	1	595
The Experimental Estuary Water Treatment Plant	1	613
Administrative Steps to a Regional Water Reuse Study: The Orange and Los Angeles Counties Water Reuse Study	1	634
Use of Reclaimed Wastewater to Operate a Seawater Intrusion Control Barrier	1	639
A Formula for Success	1	663
AQUACULTURE, WETLANDS AND ECO-SYSTEMS	1	669
The Solar Aquacell AWT Lagoon System For The City of Hercules, California	1	671
Recycle and San Diego	1	681
Reclamation and Reuse of Wastewater in the Suisun Marsh, California	1	685
State-of-the-Art of Water Recycling in Fish Culture Facilities	1	703
A Combination of Aquatic and Terrestrial Ecosystems for Maximal Reuse of Wastewater	1	714
Wetlands Creation Using Secondary Treated Wastewater	1	727
Potential of Aquaculture for Reclamation of Municipal Wastewater	1	740
Pilot Studies of the Solar Aquacell Controlled Aquaculture Process for Wastewater Reclamation	1	747
Reuse of Municipal Wastewater by Volunteer Fresh-Water Wetlands	1	762
INTERNATIONAL DEVELOPMENTS	2	783
Studies on Water Reuse in the Netherlands	2	785
Dan Region Project in Israel: From Laboratory Experiments to Full-Scale Wastewater Reuse	2	808
The Dan Region Wastewater Project - Stage II: An Advanced Single Stage Biological Wastewater Treatment and Renovation System	2	830
Development of Two Sewage and Solid Waste Treatment Systems with Potential for Water Reuse and Energy Recovery	2	837
Reuse of Wastewater in South Africa--Research and Application	2	886

TABLE OF CONTENTS CONTINUED

	Volume	Page
The Reclamation of Industrial/Domestic Wastewaters	2	907
Reuse of Water in Australia	2	925
Wastewater Reuse--The CCMS Drinking Water Study	2	937
The Role of Reuse in Hong Kong as a Potential Future Water Resource	2	948
Reclamation of Wastewater Treatment Plant Effluent For High Quality Industrial Reuse in Saudi Arabia	2	958
Ecologically Balanced Wastewater Renovation System	2	993
Reduction of Potable Water Demand in Arctic by Reuse	2	1009
Water Recycling--Necessity and a Profit	2	1020
A Water Reuse Program on St. Croix, Virgin Islands	2	1023
Compulsory Reuse of Water Due to Very Strict Water Pollution Control Regulations in Salisbury, Rhodesia	2	1035
INSTITUTIONAL FACTORS	2	1049
An Assessment of Legal and Institutional Barriers to Planned Reuse of Water in the Colorado River Basin	2	1051
Public Participation in the Adoption of Wastewater Reclamation Projects	2	1066
A Water Utility's Considerations for Implementing A Wastewater Reuse Dual Water System	2	1075
East Bay Dischargers Authority Reuse Survey	2	1086
Institutional Barriers to Wastewater Reclamation and Reuse	2	1099
Economics of Water Reuse in Metropolitan Water Supply and Wastewater Management	2	1139
The Costs for Spray Irrigation and Land Disposal of Municipal Wastewater in California	2	1151
Effluent Reuse in a Semi-Arid Region of the United States	2	1175
A Planned Analysis of the Political Tensions Affecting Planned Reuse of Water in the Colorado River Basin	2	1190
Legal Innovations to Promote Planned Reuse of Water in the Appropriation Doctrine States of the American Southwest	2	1203
Educational and Social Factors Affecting Public Acceptance of Reclaimed Water	2	1219
Reclaimed Water vs. Fresh Water--From The User's Viewpoint	2	1232
The Potential for Municipal Wastewater Reuse by Industry in Massachusetts	2	1235

TABLE OF CONTENTS CONTINUED

	Volume	Page
TREATMENT R&D/OPERATIONS	2	1247
Reliability of Advanced Wastewater Treatment	2	1249
Human Factors in Design and Operation of Water Reuse Facilities	2	1269
Multipurpose Water Treatment System Development	2	1276
Water Reclamation for Reuse in Northern Virginia	2	1311
Treatment Technology for Water Reuse	2	1328
Feasibility of Treating Hospital Wastewaters by Lime and Pressure Ozonation	2	1352
Wastewater Reuse--Persistence of Chemical Pollutants	2	1368
Sonics and Electrostatics--An Innovative Approach to Water and Waste Treatment	2	1390
Open-Channel Ultrafiltration Spiral Module for Water Reuse	2	1417
Quaternary Ammonium Resins as Contact Water Disinfectants	2	1425
Tahoe-Truckee Water Reclamation Plant--First Year in Review	2	1435
Regeneration of Wastewaters for Reuse Through Hydroperm Microfiltration	2	1446
Reductive Degradation Treatment of Industrial and Municipal Wastewaters	2	1487
Soda Ash Treatment of Acid Mine Drainage Impregnated Streams for Potable Water Use	2	1498
Nitrogen Removal in Modified Subsurface Sewage Disposal System	2	1517
Ozone Applications: A Water Reuse Review	2	1544
Composite Membrane for Wastewater Reclamation	2	1587
Ozone--Ultrasound Treatment of a Hospital Wastewater for Reuse	2	1596
Renovation of Municipal Wastewater by Reverse Osmosis: A Case Study	2	1614
MUNICIPAL REUSE	3	1627
The Application of Non-Potable Water Systems to Residential Service	3	1629
Impact of Pretreatment Regulations on Reuse Potential of Municipal Wastewaters	3	1648
Irvine Ranch Water District Turns Research to Application	3	1657

TABLE OF CONTENTS CONTINUED

	Volume	Page
Reclaimed Water Distribution System Planning--Walnut Valley, California	3	1663
St. Pete, Florida Implements Reuse	3	1680
Reuse Planning at East Bay Municipal Utility District	3	1688
Three California Water Reclamation Case Histories	3	1693
Closed Loop Recycle and Reuse of Water Closet Flush Water	3	1737
Recreational Area Water Reuse System	3	1755
Recreational Reuse at Lancaster, California	3	1762
MODELING, MONITORING AND INSTRUMENTATION	3	1777
On-Line Automated Water Quality Monitoring	3	1779
Water Reuse Management Using Dual Water Supply	3	1800
Systems Analysis Techniques Applied to Water Reuse Planning	3	1816
Design/Performance Analysis of Water Reclamation Facilities	3	1835
Instrumentation for Automating Water Processing Systems	3	1843
Wastewater Reuse and Water Conservation at U.S. Army Installations	3	1865
Water Reuse -- The Air Force Experience	3	1877
Ground Water Recharge with Wastewater--Heavy Metal Movement	3	1880
Development of a Rapid Coliform Detection Procedure Suitable for Monitoring Renovated Wastewater	3	1895
Data Acquisition, Monitor and Control System For a Water Reuse Pilot Plant	3	1900
Rapid Detection of Bacteria in Treated Wastewaters	3	1917
Virus Concentration from Waters by Hollow Fiber Ultrafiltration--the Importance of Hydrodynamics	3	1925
WATMAN: An Urban Water Management Model	3	1949
Evaluation of Two Wastewater Reuse Alternatives by Simulation	3	1965
Monitoring Organic Loading With the Electrochemical Organic Content Analyzer	3	1975
Dynamic Simulation in the Design and Analysis of Water Reuse Units	3	1990

TABLE OF CONTENTS CONTINUED

	Volume	Page
AGRICULTURAL REUSE	3	2007
Food Crop Irrigation With Reclaimed Municipal Wastewater	3	2009
Reuse of Sewage Effluent in Urbanizing Irrigated Valleys--Arizona's Salt River Valley	3	2026
Plant Pathogen Contamination in Reused Irrigation Wastewater	3	2058
Irrigating Private Land With Wastewater Effluent	3	2046
Vision of Balance--A City Eco System	3	2054
Water Reuse for Irrigation--Gilroy, California	3	2060
Increasing Water Supply for Home Irrigation	3	2068
Reclaimed Wastewater as a Feasible Water Resource for Landscape and Orchard Irrigation	3	2082
Accelerated Surface Water Eutrophication From Land Disposal of Sewage	3	2104
HEALTH AND QUALITY CONSIDERATIONS	3	2121
Wastewater Reclamation in California	3	2123
The Detection of Human Cellular Toxicity Due to Environmental Substances	3	2135
Health Effects of Groundwater Recharge	3	2146
Mutagenic Activity and Trace Organics in Concentrates From AWT Plant Effluents	3	2170
Biological Evaluation of Toxic Effects of Organic Contaminants in Concentrated Recycled Water	3	2187
Die-Away Kinetics of Aerosolized Bacteria From Sprinkler Applications of Wastewater	3	2196
Occurrence of Mutagens/Carcinogens in Municipal Wastewaters and Their Removal During Advanced Wastewater Treatment	3	2209
Integrated Use of Bioassays and Chemical Analyses to Evaluate the Quality of Reuse Water	3	2230
Chemical Analysis and Toxicity of Non-Volatile Organic Compounds in Wastewater	3	2239
Aerosol Generation in Sprinkler Irrigation	3	2248
Evaluation of Health Effects Data on the Reuse of Shower and Laundry Waters by Field Army Units	3	2252
Trace Element Analysis of Several AWT Plant Effluents	3	2269

TABLE OF CONTENTS CONTINUED

	Volume	Page
A Laboratory Study of Virus Uptake By Minerals and Soils	3	2274
Virus Removal from Sewage During High Rate Land Filtration	3	2282
Behavior of Volatile and Extractable Organics in Combined Biological/Physical-Chemical Treatment of Municipal Wastewater	3	2298

AUTHOR/CO-AUTHOR INDEX

NAME	VOLUME	PAGE
ABBOTT, Warren T.	3	1990
ABRAMS, Edwin F.	1	448
ACKERMAN, Roy A.	1	462
ACKERMAN, Roy A.	3	1755
ALSTEN, Christopher R.	1	747
AMANS, Larry C.	3	2046
ANDREOLI, Aldo	2	1517
ARANT, Gary	3	1663
ARGO, David G.	2	1249
AUKAMP, Donald R.	1	613
AVENDT, Raymond J.	3	1835
BALES, Roger C.	3	1663
BANKS, Harvey O.	2	1232
BANKS, Harvey O.	3	1816
BARTILUCCI, Nicholas	2	1517
BASU, Amit	2	993
BATES, John E.	1	565
BAUER, David H.	1	167
BAY, Rick W.	3	2038
BELFORT, Georges	3	1925
BENEDEK, A.	2	1009
BENNETT, Edwin R.	1	105
BENOIT, Gary B.	2	1151
BERZINS, A.	3	2104
BEVIS, Frederick	1	762
BIEDERMAN, Edmund M.	3	1663
BOONE, Gregory H.	1	167
BOUWER, Herman	3	2026
BROCK, Dan A.	3	1965
BROMLEY, D.	2	1009
BROOKS, Richard	3	1779
BROWN, Larry D.	3	1965
BRUWOLD, William H.	2	1066
BRYAN, Edward H.	1	59
BUROS, O. K.	2	1023
BURTON, Thomas M.	1	714
BUTLER, James A.	2	1498
BUTTERFIELD, Ossian R.	2	1435

NAME	VOLUME	PAGE
CAIRO, Patrick R.	2	1075
CARLSON, Timothy J.	2	1175
CAUFIELD, James D.	1	703
CEDERQUIST, Norman W.	1	685
CHAKRAVERTY, Chiranjit	1	398
CHAN, Deh Bin	1	474
CHAN, Deh Bin	1	506
CHAPMAN, Robert L.	2	1435
CHASE, William L., Jr.	1	663
CHEN, Ching-lin.	3	2146
CHIAN, Edward S.K.	1	246
CHIAN, Edward S.K.	1	506
CHRISTIAN, Robert T.	3	2230
CLARK, C. Scott	3	2230
CLAUNCH, Robert W.	1	433
CLEMENTS, Ernest V., III	1	405
CLEMENTS, Ernest V., III	3	1865
CLINE, Neil M.	1	139
CODY, Terence E.	3	2230
COGLEY, David	3	2252
COHEN, A.	2	808
COOPER, William J.	1	10
COSENTINO, I.A.	3	1755
COWEN, William F.	3	1975
CRAFTS, Colleen M.	1	639
CROOK, James	3	2123
CROOK, James	2	1066
CROSBY, Samuel C.	1	462
CULP, Gordon	1	595
CULP, Gordon	2	1328
CULP, Russell	1	595
CUNDELL, Anthony M.	3	1895
CUSHNIE, George C., Jr.	1	323
DAVENPORT, Ronald J.	3	1975
DAYTON, M.J.J.	3	2104
DEB, Arun K.	2	1075
DEB, Arun K.	3	1800
DEB, Arun K.	3	1949
DeKERNION, Paul S., III	2	1390
DEMGEN, Francesca C.	1	727
DE VITO, John E.	1	271
DONOVAN, John F.	1	565
DRYDEN, Franklin D.	3	1762
DRYDEN, Franklin D.	3	2146
DUFFER, William R.	1	740
DUNCAN, James H.	2	1446
DUYNSLAGER, William A.	3	1680
DYER, Jon C.	3	1648
EARLES, James D.	3	2046
EATON, James C., Jr.	1	10
EATON, James C., Jr.	3	2252
ELIA, Victor J.	3	2230
ELLINGWOOD, R. E., Jr.	1	462
ENGLANDE, A. J., Jr.	2	1368
ENGLISH, John	1	167

NAME	VOLUME	PAGE
ESVELT, Larry A.	1	336
EVEREST, William R.	3	2082
FAISST, Justine A.	2	1328
FARCHILL, David J.	2	830
FINDL, Eugene	3	1895
FLOOD, Francis J.	1	199
FONG, Michael A.	1	639
FORD, M. E., Jr.	2	1629
FORGIONE, Robert	2	1517
FOWLER, Lloyd G.	1	179
FOWLER, Lloyd G.	3	2060
FOY, Wesley	3	2252
FOX, K.	1	301
FOX, Ronald L.	2	1587
FRIEDMAN, R.	2	808
FUHS, G. Wolfgang	3	2274
GAGE, Stephen J.	1	41
GEINOPOLOS, Anthony	1	283
GERBA, C. P.	3	2282
GILLEN, James S.	1	199
GOLDSTEIN, D. J.	1	371
GRAY, Sam A.	3	1965
GREGG, John S.	1	271
GRUENER, Nachman	3	2187
GUNN, George A.	2	1311
GUNZ, Marvin	2	1425
GYORKI, John R.	3	1900
HAIS, Alan B.	1	48
HALL, Philip G.	1	585
HAMMER, LeAnne	1	405
HAMMER, Mark J.	3	2038
HARLIN, Curtis C., Jr.	1	740
HARNETT, John S.	1	585
HART, Herbert H.	1	336
HAZEL, Gary J.	1	546
HEGENBART, Joseph L.	1	634
HENNING, J. A.	2	1219
HICKS, R. Edwin	1	371
HIGGINS, Thomas E.	3	1880
HILL, A. G.	2	1352
HILLMAN, Richard L.	3	1949
HINRICHS, Daniel	1	595
HORNE, F. Wiley	1	546
HOWELL, J. B.	2	1352
HRUBEC, J.	2	785
HSIAO, Joyce S.	3	2248
HUANG, Ju-Chang	1	260
HUGHES, Mark V., Jr.	1	85
HUNTER, Robert M.	3	1693
HURD, Dale R.	2	1151
HURST, William F.	3	1657
IDELOVITCH, E.	2	808
IMEL, Charles E.	1	474

NAME	VOLUME	PAGE
JANAUER, Gilbert E.	2	1425
JAQUES, Robert S.	3	2009
JOHNSON, Charles C., Jr.	1	613
JORLING, Thomas C.	1	36
KALINSKE, Anton A.	2	958
KATZENELSON, Elie	3	1925
KING, Darrell L.	1	714
KING, Richard W.	1	681
KIRKPATRICK, William R.	3	2009
KITCHENS, Judy F.	3	1917
KNORR, Daniel P.	1	212
KOHLMANN, Harold J.	1	295
KOSAREK, Louis J.	1	421
KRUSE, Wayne E.	1	634
KU, Chia-Soon	1	448
KUEPPER, Theodore A.	1	474
LALIA, Madeline	2	1425
LAMBERT, Walter P.	2	1269
LAMBERT, Walter P.	3	1843
LAMBERT, Walter P.	3	1900
LANCE, J. C.	3	2282
LARKIN, Donald G.	3	1688
LARKIN, John F.	1	496
LAU, To-Hin	2	948
LAUGHTON, Richard V.	2	837
LEE, Garry E.	2	1086
LEE, Garry E.	3	1657
LEE, John S.	1	167
LEE, Myron K.	2	1276
LEFTWICH, D. Blair	2	1390
LENKEVICH, Michael V.	2	1498
LIGHT, William	3	2252
LIM, Lam K.	1	48
LINSTEDT, K. Daniel	1	105
LITSKY, Warren	2	1235
LIU, Han-Lieh	2	1446
LOHMAN, Loretta C.	2	1051
LOHMAN, Loretta C.	2	1190
LONG, David A.	2	1498
LOPEZ, J. G.	2	1614
LUTHY, Richard G.	1	522
MADANCY, Robert S.	1	28
MANKES, Robert O.	1	239
MARCUS, Susan	2	1425
MARIENTHAL, George	1	6
MARSHACK, R. A.	2	1219
MARTIN, Guy R.	1	22
MARTIN, Stephen R.	2	958
MASON, Marcus	3	2252
MC CARTHY, J. J.	2	1352
MC CARTY, Perry L.	2	1249
MC CUTCHAN, J. W.	2	1614
MC KENDRICK, John	2	1035
MC NUTT, Gary R.	3	1877

NAME	VOLUME	PAGE
MELTON, Robert G.	3	2170
MENDOLA, Dominick	1	671
MENDOLA, Dominick	1	747
MICHAIL, M.	2	808
MILLER, Roy D.	2	1269
MILLIKEN, J. Gordon	2	1051
MILLS, Robert L.	3	1693
MOORE, Rebecca S.	3	2274
MORGAN, James T.	1	167
MORRIS, Phillip J.	3	2248
MOZUMDER, Sashi K.	2	1139
MURPHY, Daniel F.	2	1086
NAITO, Sachiho	2	1021
NEAL, M. W.	3	2209
NELLOR, Margaret H.	3	2146
NIBLEY, David	3	1779
NISHIOKA, Kenji	3	1779
NUTE, J. Warren	1	727
ODENDAAL, Peter E.	2	886
OLIVA, James A.	1	199
OLSON, Betty H.	2	1219
O'NEILL, Patricia E.	3	1917
OSANTOWSKI, Richard A.	1	283
PAHREN, Herbert R.	3	2170
PAHREN, Herbert R.	3	2269
PARKER, Clinton E.	3	1737
PAUL, Robert A.	3	2082
PERRINE, Richard L.	2	1099
PETERMAN, Barry	1	10
PETERMAN, Barry	3	1843
PETERMAN, Barry	3	1975
PISANI, Anna M.	3	1895
PISANI, J. A.	2	1417
POPKIN, Barney P.	3	2068
PORTER, Janet E.	3	1895
POUSCHINE, Ivan, Jr.	1	295
POWELL, J. David, Jr.	3	1843
PRATTE, Janice	2	1235
PRENDIVILLE, Paul W.	1	565
PRESSLEY, Thomas A.	3	2298
RADIMSKY, Jan	1	69
REDDY, Michael M.	3	2274
REIMERS, Robert S. III	2	1368
REIMERS, Robert S. III	2	1390
REINHARD, Martin	2	1249
REUTER, LeRoy	1	3
REUTER, LeRoy	2	1276
REYNOLDS, John W.	3	1737
REYNOLDS, Royal	2	1517
RICHARDS, R.	2	937
RICHARDS, William G.	2	1075
RIGBY, M. G.	2	1219
RISSMANN, Edwin F.	1	448

NAME	VOLUME	PAGE
ROBBINS, Millard H., Jr.	2	1311
ROCHE, W. Martin	1	685
ROMAN, William	3	1779
ROSEN, Harvey M.	2	1544
ROSENBERG, Myron S.	3	1816
ROTEM-BORENSZTAJN, Yael	3	1925
ROTH, T.	2	808
ROTHBERG, Michael R.	1	105
ROY, Dipak	1	246
SANDERSON, Jack T.	1	55
SANTO, John E.	2	1446
SAXENA, J.	3	2209
SCHERER, Charles R.	2	1139
SCHERER, Clarence H.	1	467
SCHERFIG, Jan	3	1657
SCHIPPERS, J. C.	2	785
SCHMIDT, Curtis J.	1	405
SCHMIDT, Curtis J.	3	1865
SCHWARTZ, D. J.	3	2209
SERFLING, Steven A.	1	671
SERFLING, Steven A.	1	747
SHAIKH, A.M.	2	937
SHAPIRA, N. I.	2	1446
SHEIKH - OL - ESLAMI, Bahman	3	2009
SHEIKH - OL - ESLAMI, Bahman	3	2248
SHELTON, Stephen P.	3	1865
SHUVAL, Hillel	3	2196
SIERKA, Raymond A.	2	1596
SKAGGS, Robert L.	2	1596
SKELLY, Michael J.	1	496
SMALL, Mitchell J.	3	1900
SMITH, Cecil L.	3	1990
SMITH, Duncan K.	2	837
SMITH, Michael A.	2	925
SMITH, Stanley A.	2	1435
SNELL, John R.	2	1021
SOLOMONS, Clive C.	3	2135
SPATH, David P.	3	2123
STARKS, David M.	3	1990
STEADMAN, James R.	3	2038
STEELE, Myron E.	3	1693
STENQUIST, Richard J.	3	1693
STERN, K. M.	2	937
STEWART, William C.	1	747
STOKES, H. W.	3	1629
STURMAN, Lawrence S.	3	2274
SUDAK, Richard G.	2	1587
SUTHERLAND, Jeffrey C.	1	762
SWAYNE, Michael D.	1	167
SWEENY, Keith H.	2	1487
TADMORE, Jacob	3	2196
TALJARD, Mark P.	2	907
TAYLOR, Dene H.	3	2274
TETSCH, Benjamin	3	2196
THOMAS, Richard	3	1779

NAME	VOLUME	PAGE
THOMAS, Richard E.	1	48
THOMPSON, Harold W.	1	336
THURBER, Marvin D.	3	2054
TOBIN, Patrick	1	192
TREWEEK, G. P.	1	301
TRUMBLY, Anthony S.	2	1203
TRUSSELL, R. R.	1	301
TUNNAH, B. G.	2	937
TURNER, Ronald J.	1	448
ULMER, Nancy S.	3	2269
VAJDA, Gary F.	1	522
VALENTINE, Ralph S.	3	1917
VAN VUUREN, Lucas R.	2	886
VAN VUUREN, Lucas R.	2	907
VELTEROP, F. M.	2	1417
WALFISH, Ilona H.	2	1425
WALTERS, Richard W.	1	522
WALTON, Harold F.	3	2239
WASSERMANN, Kurt	1	69
WEDDING, Jerome J.	1	639
WEI, Irvine W.	1	371
WENTSEL, Randall	3	1917
WESNER, G. M.	1	85
WESNER, G. M.	1	192
WILLIAMS, Robert B.	1	595
WILLIAMS, Robert B.	2	1328
WILLIS, John F.	2	958
WINDOLPH, Gary R.	2	1175
WOJCIK, C. K.	2	1614
WOOD, William W., Jr.	2	1151
WORK, Stephen W.	1	105
YANG, Patrick Y.	3	1843
YANG, Patrick Y.	3	1900
YOSHIMURA, Robert Y.	1	634
YOUNGNER, Victor B.	2	1151
ZOETEMAN, B.C.J.	2	785

WATER REUSE
SYMPOSIUM

INTERNATIONAL DEVELOPMENTS

STUDIES ON WATER REUSE IN THE NETHERLANDS

J. Hrubec
National Institute for Water Supply
Voorburg, The Netherlands

J.C. Schippers
The Netherlands Waterworks Testing
and Research Institute KIWA N.V.
Rijswijk, The Netherlands

B.C.J. Zoeteman
National Institute for Water Supply
Voorburg, The Netherlands

INTRODUCTION

The Netherlands (figure 1) is a country rich in water. It is also a highly industrialised area with one of the world's highest population densities.

Figure 2 illustrates the rapid increase in demand for public water supply from the IInd World War up to the early seventies.

According to a prognosis of 1972 a similar rapid increase of water consumption till the year 2000 was expected. However, lower population growth, changes in the water consumption pattern both in households and industry, and a retarded economic growth, in 1978 resulted in a modified prognosis with a much smaller rate of increase of the water consumption.

The reduction in consumption of water in industry since the early seventies (figure 3), is mainly caused by a decrease of water consumption per unit of production, which was encouraged by the new Water Pollution Control Act which came into force in 1970.

Under this Act a system of levies for discharge of waste water was introduced leading to a limitation of the volume of discharged waste water in the industry and consequently to a reduction of the water consumption.

At present about two thirds of water for domestic and industrial purposes in The Netherlands is derived from underground sources and one third from surface water sources. As the exploitation of the underground sources practically is approaching the capacity of the aquifers in the country the additional water demand mostly has to be covered from other sources.

The most important source of surface water in the country is the river Rhine (table 1). The river is strongly polluted. Production of drinking water from the Rhine, therefore, may be considered as indirect reuse of waste water for human consumption. The pollution is caused mainly by discharges of untreated or only partly treated sewage water from a catchment area with a population of about 60 million inhabitants and by discharges of wastes from chemical industries. A consequence of these discharges is e.g. the pollution of the river with refractory organic compounds. During the early part of the seventies the load with difficult degradable organic compounds increased yearly with 6-8 %. Stumm and Roberts (1) predicted that the load of refractory organic compounds from the catchment area of the river would increase from around 240.000 ton carbon in 1970 to 500.000 ton in 2000, in spite of biological treatment of all municipal and industrial waste water. Furthermore, the chloride and sodium concentrations in the river are steadily increasing (figure 4) as a result of the discharge of salt

eceding page blank

wastes from the mine industry and other more diffuse sources. The load of chloride in the river Rhine at the Dutch-German border increased from approximately 175 kg/sec in 1950 to more than 350 kg/sec in 1975 and the concentration of chlorides of the river water during the low flow in 1975 was higher than 250 mg/l.

The sodium concentration is coupled with chlorides and hence increasing also. Due to the acceptance of the new drinking water standard of the European Community for sodium (175 mg Na/l) this constituent will become one of the critical parameters for the quality of the river Rhine.

To overcome periods during which the quality of the river water does not meet the required quality standards for preparation of drinking water, water storage in open or underground reservoirs is necessary.

The realisation of these reservoirs which are often located in areas of great ecological value is a subject of growing public criticism.

Substantial improvement of the water quality of the river Rhine could be achieved by intensive water pollution control within the whole catchment area. However, an effective pollution control is a difficult international political problem which recently has been partly regulated in a Treaty on the Chemical Pollution of the Rhine. Up till now no appreciable improvement of the quality of the riverwater has been achieved.

Taking into consideration climatological, geographical and economical aspects the following supplementary alternatives within the country for the water supply seem to be most suitable:

- limitation of the use of groundwater for industrial and agricultural purposes by means of the use of surface water or by means of direct or indirect reuse of industrial and municipal waste water;
- use of desalinated brackish groundwater for potable water supply;
- direct or indirect reuse of municipal waste water for nonpotable domestic use (e.g. reuse in buildings, university complexes etc.).

Reuse of municipal waste water is a possible alternative for the present indirect reuse via the river Rhine.

At a regional scale the reuse of municipal waste water for artificial groundwater recharge destined for drinking water supply and for salt intrusion control, provides an alternative for the water supply of the islands located along the Dutch northwest coast ("Waddeneilanden").

These islands suffer from a serious shortage of drinking water during the recreational summer season, when the population number is multiplied several times.

To evaluate the possibilities of the water reuse a programme of studies on different aspects of direct and indirect reuse was encouraged in the early seventies.

One of the objectives of the studies is to evaluate the efficiency of the available waste water technologies especially with respect to the removal of potential hazardous materials and to quantify the risks involved in the extreme case of direct reuse of municipal waste water for potable and non potable purposes and in the indirect reuse via the river Rhine.

The investigation probably will provide data for the formulation of quality criteria for different water sources and different applications of reused water. These criteria are necessary to enable further evaluation and for planning purposes.

The investigations are carried out in a pilot plant in Dordrecht.

In this paper the facilities and preliminary results are described and the future research programme is indicated.

DORDRECHT PILOT PLANT

The pilot plant is located on the site of the Dordrecht municipal wastewater treatment facility. The feed is secondary effluent of the activated sludge plant that treats wastewater, mostly of domestic origin, from the city of Dordrecht.

The industrial wastes of the major industries in the area are treated separately and then discharged directly into the river system.

The Dordrecht waste treatment plant consists of grit removal, primary sedimentation, activated sludge process and secondary sedimentation.

About 10 % of the effluent from the plant is recycled for scrubbing flue gas from the incineration plant for household refuse and sludge from the wastewater treatment plant. The heated wash water containing ash, HCl, HF and H_2SO_3 is mixed with the raw wastewater influent following neutralisation and sedimentation.

The secondary effluent was monitored during 1975/76 to obtain information on seasonal fluctuation in quality. Fluctuations in COD and some heavy metal concentrations (Cd, Pb) are shown in figures 5, 6 and 7.

Fluctuations in quality are especially due to rainfall when mixing of raw water with runoff water occurs.

Typical quality characteristics of the secondary effluent are presented in table 2.

Figure 8 shows a chromatogram of Dordrecht secondary effluent in comparison with that of Rhine water and illustrates the fact that the concentrations of many synthetic organic compounds are higher in the river water then in the biological effluent.

In 1978 mutagenicity of unconcentrated samples of the secondary effluent was determined six times by means of the Ames test. In no sample mutagenic response was detected, while the Rhine water showed significant mutagenicity with a similar determination procedure.

The pilot plant consists of two systems: a combined biological and physical-chemical treatment scheme and a system in which reverse osmosis is incorporated.

PHYSICAL-CHEMICAL SYSTEM

The physical-chemical system was selected on the basis of experience with advanced wastewater treatment in the USA and South Africa, showing that water of high quality could be produced in an effective and economic way with a combination of the various physical-chemical processes incorporated in our scheme.

Description of the system

The system is shown in figure 9.

a. Lime treatment at pH 11.2

This process was preferred over other chemical-coagulation processes for removal of phosphorus and suspended and colloidal solids because partial disinfection and more effective removal of potentially toxic trace elements can be achieved at high pH. Furthermore, in contrast to coagulation processes with aluminium and ferric salts no anions, like chlorides and sulphates, are introduced during lime treatment.

Coagulation is accomplished with lime at doses of 250 - 600 mg $Ca(OH)_2/l$ using a rapid-mixing tank and a flocculation tank followed by a sludge-blanket reactor. The retention time for the average design flow of 4 m^3/hr is 1.5 hr. The upflow rate for the sludge-blanket reactor is 1 mm/sec.

Polyelectrolyte (0.5 mg/l) is added to the influent of the flocculation tank to enhance flocculation and to improve sedimentation.

b. Recarbonation

Recarbonation for pH correction can take place after the sedimentation tank or after the ozonation columns. Hence, the possibility for ozonation at high pH exists.

First-step recarbonation is carried out in a column, 3.0 m high and 0.5 m in diameter. The hydraulic retention time is 9 minutes with a linear liquid velocity of 20 m/hr. Dosing of CO_2 is controlled automatically.

The second recarbonation unit consists of two columns, 2 m high and 0.3 m in diameter, with a etention time of 4.5 min and a water flow rate of 50 m/hr; these can be used individually, in series or parallel.

c. Double-layer filtration

The double-layer filter bed consists of an 80 cm high layer of 1.5 - 2.5 mm crushed sand and a 20 cm layer of 2 - 4 mm anthracite. The relatively course filter material was chosen to prevent rapid increase of head loss and thus to obtain longer filtration runs. The filter is operated in the downflow mode with a filtration rate of 5 m/hr. Backwashing and air scouring is controlled manually.

d. Ozonation

The primary purpose of the ozonation step is disinfection. In addition, improvements in taste, odour, colour and turbidity are expected along with partial removal of organic matter and oxidation of some organic micropollutants (phenols and polycyclic aromatic hydrocarbons).

The ozonation is carried out in three 4.0 m high columns, with a minimum retention time of 12 minutes. Gas dispersion tubes for O_3-dosing are used to obtain fine gas bubbles. The maximum O_3 dose is restricted to 15 mg per litre because of the capacity of the ozone generator.

e. Selective ion exchange and dry filtration

A combination of biological dry filtration (trickling filtration) with selective ion exchange of ammonium ion by means of clinoptilolite was chosen for removal of nitrogen compounds.

Dry filtration has been used widely in The Netherlands since the early nineteen fifties for removal of ammonia from groundwater.

Also, the dry filtration has been used with success on a pilot plant scale for removal of ammonia from river water.

The dry filtration is based on percolation of air with water through the filter bed. The aeration of the bed must be intensive enough to maintain aerobic conditions in the bed even when rapid oxygen consumption occurs during the nitrification of large ammonia concentrations. Artificial aeration of the filter bed is necessary to obtain full nitrification. In practice complete removal of concentrations of ammonia as high as 30 mg NH_4^+/l was achieved with artificial aeration.

A disadvantage of this process is a substantial decrease in removal efficiency at low water temperatures. Ammonia was only partly removed in the treatment of Rhine water at temperatures below $5^{\circ}C$.

However, such low temperatures are normally not expected for the effluents of wastewater treatment plants.

There are two dry filters in the scheme, which can be operated either parallel or in series. The depth of the bed of each filter is 1.55 m; the filter material in the first filter is river sand with a diameter of 4 - 6 mm and in the second one is crushed sand with a diameter of 2 - 3 mm. The maximum filtration rate with parallel operation is 4 m/hr and with series connection is 8 m/hr. The aeration is accomplished by blowing in air beneath the filter bottom at a rate of 8 air volumes per volume of water. There is a provision for backwashing of the filter with water (max. rate 120 m/hr) with or without air.

The zeolite filters serve as a polishing step for ammonia removal. Furthermore, it is hoped that a combination of the dry and zeolite filters can be used to prevent high concentrations of nitrate in the product water. A partial by-pass of the dry filters will be used during periods of high nitrate concentrations in the effluent of the dry filters. The two zeolite filters are operated alternately.When one is actively functioning, the other one is in reserve. Anaconda clinoptilolite with ion exchange capacity of 0.3 meq ammonia per gram at an ammonia concentration of 30 mg/l in the influent and 0.5 mg/l in the effluent has been used. The maximum velocity is 7 bed volumes of water per bed volume of the zeolite per hour.

The clinoptilolite is regenerated with 1.2 per cent solution of NaCl at pH 11.

f. Activated carbon filtration

Four 0.5 m diameter activated columns in series, each with a 1.5 m carbon bed, may be used in either upflow or downflow. The contact time at a maximum flow rate of 2.5 m^3/hr is 28 minutes.

An aeration column filled with Raschig rings is placed between the 2nd and 3nd column. The purpose of the aeration is to prevent anaerobic conditions during the last part of the adsorption process.

g. Disinfection by sodiumhypochlorite or chlorine dioxide

Final disinfection is provided by addition of sodiumhypochlorite to the effluent of the activated carbon columns. Chlorine dioxide can be used as an alternative.

The dosage is based on the attainment of residual free chlorine equal to 0.5 mg/l in the effluent of the contact tank after a retention time of 30 minutes.

Preliminary results of the lime process

The first part of the system, the lime process, has been operational since 1977, the other parts of the system are in use only since the beginning of this year.

A brief review of the results of the lime process is presented with respect to the following parameters of water quality:

(a) phosphorus and other inorganic substances; (b) organic compounds; (c) trace elements; (d) bacteria and viruses.

a. Phosphorus and inorganic substances

The efficiency of removal of phosphate and ionorganic substances is shown in table 3.

Phosphates were removed quite effectively. Especially during summer and autumn the mean concentrations in the effluent were very low (0.05 mg PO_4/l). During the severe winter of 1978/79 the raw water temperature dropped to $10^{o}C$ and a decrease in removal efficiency was observed: the average phosphate concentration in the effluent increased to 0.15 mg PO_4/l.

High degrees of removal of magnesium and of iron were achieved during the tests without iron dosing.

Calcium concentrations in the effluent showed large fluctuations; greater concentrations in the effluent than those in the influent were frequently measured. This is due to frequent overdosing of lime during periods of rapid fluctuation of the influent alkalinity, when it was difficult to maintain a constant pH.

Because of the possibility of higher fluoride concentration in the biological effluent containing wash water from the flue gas of the incineration plant, determination of fluoride was instituted in 1978. The mean fluoride removal was about 30 % and the concentration in the effluent of the sludge-blanket reactor became 0.8 mg F/l.

b. Organic compounds

Removal of organic matter, as measured by COD, was not as effective as expected. The average removal was only 25 % in 1977, for COD values of the effluent of the process equal to 20 - 30 mg O/l.

Reduction of colour-causing substances was also moderate. A positive effect of iron dosing on colour improvement was evident: during Fe-dosing colour removal increased to about 70 per cent.

Volatile halogenated organic substances and non-polar organic chlorine were determined occasionally. As expected, no removal of these components by the lime process was detected. The low concentrations of all detected trihalomethanes in the raw secondary effluent are interesting, because these were lower than the mean concentrations in the drinking water of Dordrecht. The concentrations of non-polar organic chlorine in the biological effluent (1 - 4 μg/l) are lower than in the Rhine water, which are of the order of 10 μg/l (2).

c. Trace elements

Determinations of concentrations of seven trace elements (table 4) confirmed the effective removal of heavy metals by the lime process. An exception is an anomaly in the behaviour of chromium, its concentration being greater in the effluent than in the influent. The reason was the Cr content of the lime used. In addition, the percentage of As removal was low. Nevertheless, the concentrations of all determined trace elements in the effluent water were well below drinking water standards.

d. Bacteria and viruses

The number of bacteria and viruses was determined during three months in 1977 (table 5).

The table shows that bacterial removal was fairly effective. The organism *Pseudomonas aeruginosa* seems to be most resistant to the high lime treatment.

Enteroviruses were determined in the influent and effluent water 10 times. Enteroviruses were detected in four samples of the influent and ranged to a maximum of 560 PFU per litre. The percentage removal was 35 - 100 %.

Conclusion

The data on the performance of the lime process confirmed that this process can achieve effective removal of phosphates, micro-organisms and heavy metals. The removal of organic compounds was lower than was expected.

The reliability of the process was not satisfactory. Particularly a breakthrough of sludge-blanket caused serious problems. Furthermore, due to high lime consumption the costs of the process are relatively high.

REVERSE OSMOSIS SYSTEM

The principal reasons for the incorporation of reverse-osmosis in the Dordrecht pilot plant study were the high efficiency of this system with regard to removal of almost all inorganic, organic and biological constituents and the fact that the reverse-osmosis seems to be the best available alternative for elimination of a build-up of refractory pollutants during multiple reuse.

Description of the system

In figure 10 the pilot plant scheme for the reverse-osmosis system is shown.

The pilot plant consists of:

- dosing pumps for sodiumhypochlorite, ferric chloride and polyelectrolyte;
- an upflow contact filter and a downflow rapid filter;
- dosing equipment for hydrochloric acid;
- the reverse-osmosis unit itself;
- an activated carbon filter;
- a dry marble filter;
- a final dosing pump for sodiumhypochlorite.

In the design of the pilot plant installation special attention was given to the following technical aspects:

a. keeping membrane fouling to a mininum;
b. prevention of biological attack of the membrane;
c. assurance of complete removal of ammonia, which is only partly rejected by reverse-osmosis;
d. reduction in content of organic pollutants, including low molecular micropollutants, to a minimum;
e. prevention of formation of trihalomethanes;
f. elimination of corrosivity of product water.

a. Membrane fouling

It is generally agreed that membrane fouling is an important obstacle in the application of reverse-osmosis for the advanced treatment of wastewater. The factors substantially affecting the fouling are the type of membrane and the pretreatment of the feed water. Tubular membranes which can be cleaned easily were selected for the pilot plant to limit the problem of fouling.

As pretreatment a combination of contact upflow filtration with rapid filtration was chosen. Both filters are filled with sand 1.2 - 2.4 mm. The filters are operated at a hydraulic rate of 2.5 m/hr. Ferric chloride and polyelectrolyte, Superfloc C_{573}, are applied to the influent of the contact filter. The dosages are approximately 3 mg Fe/l and 4 mg C_{573}/litre. In addition to feed-water quality and type of membrane, the hydrodynamics of the reverse-osmosis system also affect the rate of fouling. In particular, the values of the flux and of the velocity of water along the membrane surfaces play an important role.

The results of the tests with the laboratory model (figure 11) show that a relatively high velocity of water along the membranes with low salt rejection has to be applied to reach sufficient flux. A disadvantage of application of high velocity along the membrane is the large pressure drop in the membrane module, which leads to a decrease in the driving power. This aspect together with the fact that membranes with low flux exhibit higher salt rejection led to the selection of high-rejection membranes.

The following reverse-osmosis system was selected on the basis of the results of laboratory tests: Cellulose acetate tubular membranes of Paterson Candy International with salt rejection 97 %. The membranes of 1.2 cm diameter have a length of 3.5 m.

The system comprises 10 steps with the following arrangement of the modules: 5 4 4 4 4 4 3 3 3 2 2. A minimum velocity past the membranes of 1 m/sec could be maintained with this arrangement. The total membrane area is 2.6 m^2.

At the pressure of 40 bar and with a recovery ratio of 60 - 80 per cent the production capacity of the system is 1.5 m^3/hr.

b. Prevention of biological attack of membranes

Because of the relative high sensitivity of cellulose acetate membranes to biological attack, an effective disinfection of the membranes is required. Because of the lack of information on the stability of the membranes against oxidation by other disinfectants, chlorine has been used for this purpose.

However, chlorine seems to be far from an ideal disinfectant. Particularly during relatively rapid fluctuations in ammonia concentration in the secondary effluent, it is difficult to determine an effective dosage of chlorine due to the occurrence of breakpoint reactions.

Moreover, application of chlorine to water containing high concentrations of organic compounds is undesirable due to the formation of trihalomethanes, which are not removed by reverse-osmosis.

During the first phase of the pilot plant tests (January - July 1978) chlorine was added to the secondary effluent in doses up to 5 mg/l. This resulted in residual concentrations of total chlorine of 0.1 - 0.4 mg/l in the feed.

Since August 1978 periodic shock application of chlorine with a maximum dose of 40 mg/l for an hour has been used with a freque.cy of about once a day during summer and down to once a week in winter. In this way formation of trihalomethanes could be limited.

c. Ammonia removal

In spite of the relatively high rejection of ammonia with reverse-osmosis (about 90 %) the ammonia concentrations of the membrane permeate are unacceptably high for drinking water purposes. Nitrification on a dry marble filter is used for removal of the rest of the ammonia concentration.

d. Removal of organic compounds

Effective removal of organic compounds (as measured by TOC and COD) by treatment of waste water with reverse-osmosis is well documented.

The high rejection of TOC and COD can be explained by the fact that the organic substances in waste water consist mostly of high molecular compounds (humic substances, carbohydrates and proteins).

Relatively little is known about the effect of reverse-osmosis on removal of individual micropollutants, which are often compounds of low molecular weight.

Data obtained from some studies on this problem (3,4) show that rejection may be very low depending on molecular weight, polarity and structure of the molecule and on the physical-chemical properties of the membrane. An activated carbon filter was incorporated in the system to enable additional reduction of organic micropollutants passing the reverse-osmosis. This filter is placed after the reverse-osmosis, which removed a substantial part of high-weight molecular organic substances, thus leaving the adsorption capacity of the activated carbon filter be utilized by micropollutants. The filter is 0.5 m in diameter with a 1.8 m bed depth. The hydraulic rate is 7 m/hr and the contact time is 15 minutes.

e. Prevention of formation of trihalomethanes

To restrict the formation of trihalomethanes during treatment the application of chlorine prior to removal of organic matter should be limited. Because the use of chlorine for membrane disinfection is necessary, the only alternative was to limit its application by the use of shock dosing with as low a frequency as possible. The use of chlorine dioxide as alternation is a subject for future investigation.

f. Elimination of corrosivity of the product water
As result of acid addition to the feed to prevent oversaturation of $CaCO_3$ and hydrolysis of cellulose acetate the product water has a low pH and high carbon dioxide content. For conditioning of this water a combination of carbon dioxide stripping and neutralisation with calcium carbonate seems to be an attractive possibility.
This combination is realised in the dry marble filter, where, by means of the introduction of air beneath the filter, part of the CO_2 is stripped and part is neutralised with the marble. By regulating the air volumes it is -within limits- possible to achieve a desired value of pH and of hardness.
The filter is 0.5 m in diameter; the medium is 1.7 m of 1.5 - 2.7 mm granular marble. The hydraulic rate is 7 m/hr and the backflow air velocity is kept at about 10 m/hr.

Results

a. Technological aspects
In table 5 the data on flux decline and on salt rejection during 10.000 hours of continuous operation are presented. The flux is recalculated to a pressure of 40 bar at 20° temperature. The rejection is based on conductivity determinations. The salt recovery ratio over the reported period was 60 - 80 %.
The flux decline rate is typical for cellulose acetate membrane used to treat secondary effluents and gives good hope that a projected module life of three years may be achieved. The membranes are cleaned every three weeks by depressurization and flushing with foam balls. Once every two months an enzyme product is used. Up till now no sign of membrane abrasion has been observed.

b. Water quality aspects
A summary of data on effluent quality at various stages in the treatment process is presented in tables 6, 7 and 8.
In table 6 the concentrations of inorganic constituents are shown. As expected, the percentages rejection of the inorganic constituents are great with exception of those for ammonia and nitrates, for which the removals are 90 % respectively. Nearly complete elimination of ammonia is achieved with the marble dry filter.
Also the data on organic matter and trace elements, presented in table 7, indicate high rejection of all determined constituents.
Data on organic micropollutants are summarized in table 8.
Non-polar micropollutants have been isolated by means of the closed-loop stripping procedure of Grob (5) and more polar compounds by means of the XAD elution technique.
The compounds were identified by means of a GC/MS computer combination. The table shows that particularly the groups of halogenated aliphatics, benzene, C_1 and C_2 benzenes, naphthalene, alkylated naphthalenes, sulfonamides and alkylphenols are removed ineffectively by reverse-osmosis. Alkanes, alkylated benzenes with longer side chains, indane and alkylated indanes, organic phosphates, chlorophosphates, phthalates and cyclic hydrocarbons are rejected rather effectively.
By means of activated carbon filtration substantial additional removal of almost all detected micropollutants was achieved with exception of cyclic hydrocarbons.

Conclusion

Experience accumulated during the 14 months of operation of the scheme incorporating reverse-osmosis shows that this system has a satisfactory reliability and is capable of producing water of high and constant quality with an economically acceptable decline in membrane flux at a constant ratio of salt rejection. Fouling of the membranes can be overcome by an easy and economical cleaning procedure.

The analyses of the effluent water indicate that practically all determined chemical parameters meet the current quality standards set for drinking water.

FUTURE INVESTIGATIONS

Future investigations will be focused especially on the comparative study of the water quality produced with both pilot plant systems and drinking water of Dordrecht, which is produced from Rhine water. Special attention will be paid to qualification of health hazard pollutants and to determinations of viruses after concentration by means of a filter adsorption technique.

To evaluate toxicological aspects toxicity tests with amphibian *Xenopus laevis*, which has been started in the beginning of 1979, will be continued and in addition, to test mutagenicity, Ames tests will be carried out with highly concentrated water.

ACKNOWLEDGEMENT

The authors would like to express their gratitude to the German Marshall Fund for the travel grant which enabled J. Hrubec to participate in the Symposium and to present this paper.

REFERENCES

1. STUMM, W., ROBERTS, P.V. Prioritäten bei der Rheinwassersanierung, 3e IAWAR Tagung, Düsseldorf (1973).

2. KÜHN, W., SONTHEIMER, H., STIEGLITZ, L., MAIER, D., KURZ, R. Use of Ozone and Chlorine in Water Utilities in the Federal Republic of Germany, Jour. AWWA, 70:326-331 (1978).

3. SCHIPPERS, J.C., BOM, C.A., VERDOUW, J. Five years experience with a reverse-osmosis plant for the treatment of river Rhine water. Proceedings 6th Inter. Symposium, Fresh Water From the Sea, Vol. 3: 363-375, Athens (1978).

4. FANG, H.H.P., CHIAN, E.S.H. Reverse Osmosis separation of polar organic compounds in aquous solution. Env. Sci and Technol., 10: 364-369 (1976).

5. GROB, K., ZÜRCHER, F. Stripping of Trace Organic Substances. Journ. Chromatogr., 117:285 (1976).

TABLE 1

Water balance of The Netherlands for an average year (figures in $10^9 m^3$ per year)

River Rhine	69
River Meuse	8
Smaller streams	3
Precipitation (minus evaporation)	10
Total available fresh water	90

TABLE 2

Characteristics of secondary effluent(1975/76)

Parameter		
COD	35 - 100	mg/l
Ca^{2+}	75 - 135	mg/l
Mg^{2+}	12 - 36	mg/l
PO_4^{3-}	4.5 - 36	mg/l
Alkalinity	200 - 400	mg/l
Suspended solids	3 - 43	mg/l
Cd	0.2 - 7	µg/l
Cr	1 - 6.5	µg/l
Cu	3 - 42	µg/l
Hg	0.1 - 2.8	µg/l
Pb	4 - 26	µg/l
Zn	60 - 580	µg/l
Ni	5 - 68	µg/l
Li	17 - 38	µg/l

TABLE 3

Removal of some inorganic contaminants by the lime process and with addition of iron(average concentration)

Parameter	Lime treatment			Lime treatment with Fe-dosage (5 mg Fe^{3+}/l)		
	Influent mg/l	Effluent mg/l	Efficiency %	Influent mg/l	Effluent mg/l	Efficiency %
COD	37	29	22	51	35	31
Magnesium	23	8.4	63	24.7	15.7	36
Calcium	95	79	17	78	58	26
Iron	0.25	0.03	88	0.22	0.25	+14
Orthophosphate	14.3	0.10	99.3	29	0.05	99.8
Colour	79	60	24	82	20	76
Sulfate	101	99	2	-	-	-
Silicate	9.9	7.7	22	12.2	3.8	69

TABLE 4

Removal of trace elements with the lime process (average concentration)

Constituent	Before chemical treatment µg/l	After chemical treatment µg/l	Efficiency %
As	2.3	1.9	17
Cd	1.1	0.4	64
Cr	1.6	2.8*	-
Cu	7.5	2.6	65
Mn	331	23.5	93
Pb	7.0	1.8	74
Zn	119	10	92

* introduced by lime slurry

TABLE 5

Removal of micro-organisms by high lime treatment

Organism	Concentration (10^3 counts/ml)		Concentration (10^3 counts/ml)		Average efficiency
	Influent		Effluent		%
	Min	Max.	Min.	Max.	
Coliphages*	1.2	100	0	1.5	97.3
Coliforms	43.000	640.000	25	22.000	98.9
E. coli	380	290.000	3	460	97.0
Ps. aeruginosa	1.100	30.000	8	3.200	94.5
Clostridia	9	630	0.2	19	97.3
Fecal streptococci	1.700	5.100	1.2	160	98.0

TABLE 6

Course of waterflux and salt rejection of the reverse-osmosis

Time	200 hr.	6000 hr.	10000 hr
Flux ($m^3/m^2.d$)	0.56	0.50	0.47
Rejection (%)	97.4	97.9	97.3

TABLE 7

Removal of inorganic constituents during the Reverse-Osmosis System

Parameter *	Secondary effluent	Feed R.O.	Effluent R.O.	Effluent marble filter
Conductivity	1220 μS/cm	1220 μS/cm	50 μS/cm	215 μS/cm
Cl^-	-	161	12	-
SO_4^{2-}	-	107	2	-
F^-	0.6	0.5	0.07	0.05
HCO_3^-	365	357	15	128
pH	7.5	7.5	5.2	7.9
Ca^{2+}	102	104	3	43
Mg^{2+}	15	14	1	1
Na^+	110	110	3.5	3.5
K^+	17	16	1	1
NH_4^+	-	14.5	1.4	0.18
NO_2^-	1.0	1.0	0.01	0.01
NO_3^-	30	30	3.0	7.3
PO_4^{3-}-ortho	15	11	0.07	0.04
SiO_2	25	21	1	5

* units in mg/l unless otherwise stated

TABLE 8

Removal of organic compounds and trace elements
during the Reverse-Osmosis System

Parameter *	Secondary effluent	Feed R.O.	Effluent R.O.	Effluent marble filter
COD	47 mg/l	32 mg/l	3 mg/l	2 mg/l
DOC	14 mg/l	11 mg/l	1.7 mg/l	1.3 mg/l
As	5	4	0.5	0.5
Cd	1.2	0.4	0.1	0.1
Cr	2.4	1.8	0.5	0.5
Cu	3.0	1.8	0.5	0.5
Hg	0.1	0.1	0.1	0.1
Mo	7	7	2	2
Ni	6.3	6.8	1	1
Pb	9.4	3.3	0.5	0.5
Se	2	2	2	2
Zn	80	40	10	10

* units in μg/l unless otherwise stated

TABLE 9

Removal of organic micropollutants during the Reverse-Osmosis System

Compounds μg/l	Secondary effluent	Feed R.O.	Effluent R.O.	Effluent marble filter
Halogenated aliphatic hydrocarbons	21	8.5	11	1.1
Benzene + C_1+C_2 alkylbenzenes	0.3	0.3	0.25	0.1
Sulphonamides	3.3	3.3	3.1	-
Alkylphenols	0.69	0.69	0.31	0.36
Naphthalene + alkylnaphthalenes	0.2	0.1	0.05	0.01
Hydronaphthalenes	0.12	0.19	0.06	-
Alkanes	0.2	0.9	0.05	0.05
C_3-C_8 alkylbenzenes	5.7	3.36	0.76	0.13
Indane + alkylindanes	0.75	0.75	0.05	-
Phosphates + Chlorophosphates	0.5	0.5	0.06	-
Phthalates	10.2	1.7	0.23	0.13
Cyclic hydrocarbons	1.13	1.13	0.11	0.2

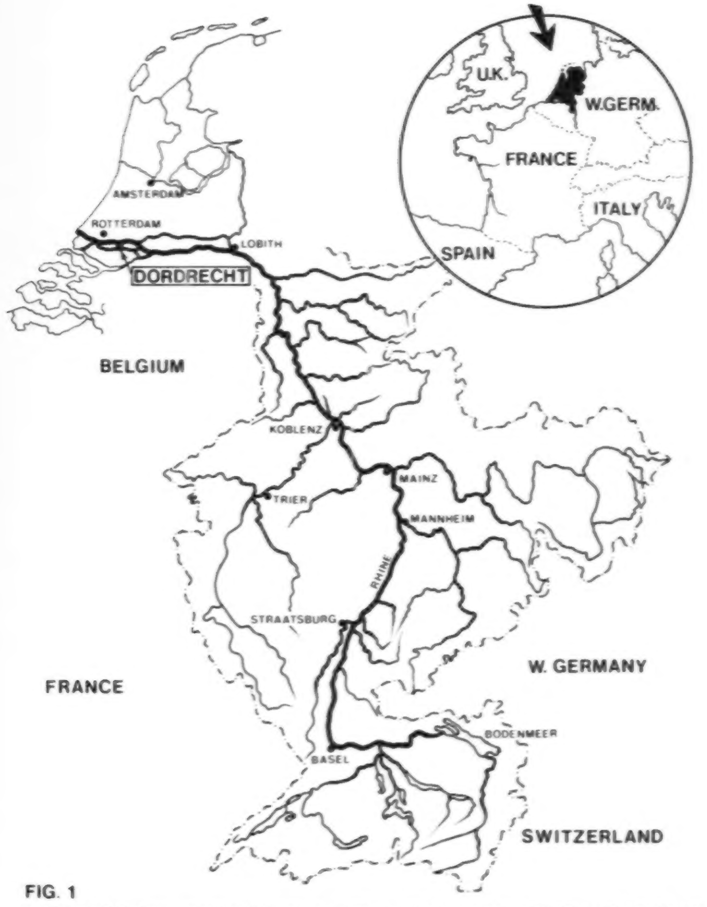


FIG. 1
HYDROLOGICAL MAP OF THE NETHERLANDS AND CATCHMENT AREA OF THE RIVER RHINE

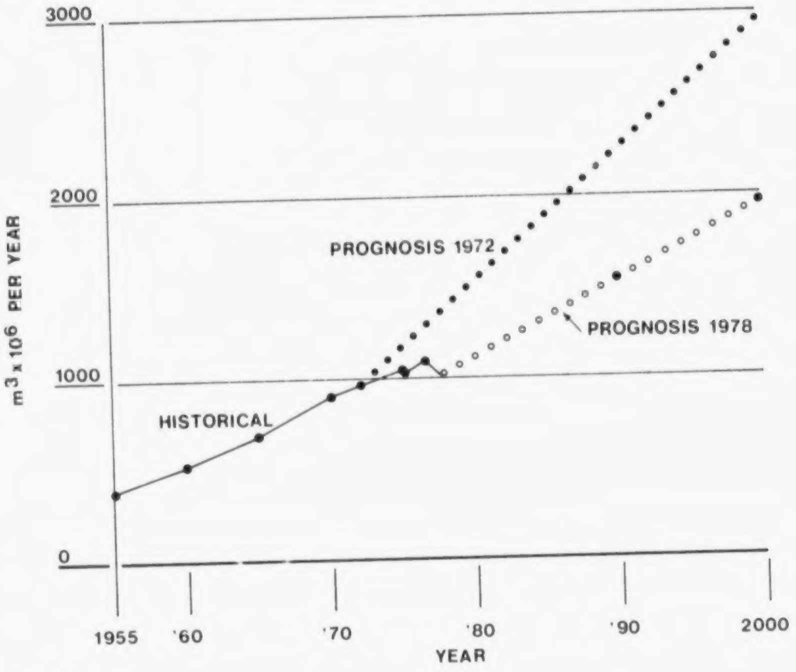


FIG. 2
WATER PRODUCTION OF PUBLIC WATER WORKS

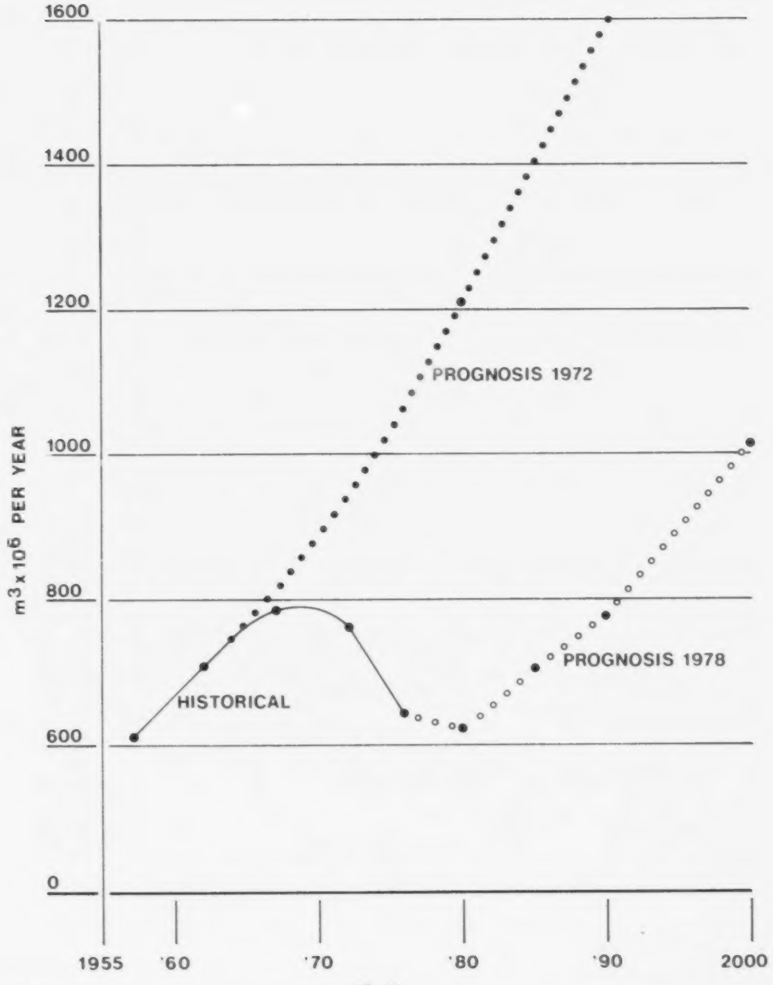


FIG. 3
DEMAND OF INDUSTRIAL WATER (EXCL. COOLING WATER FROM SURFACE WATER)

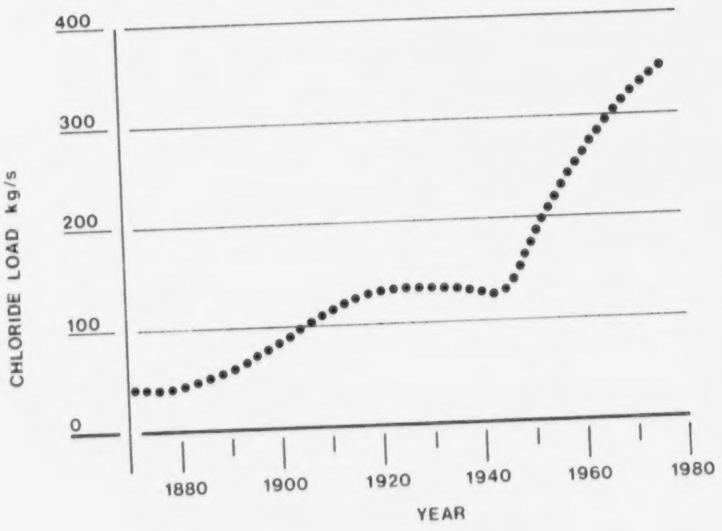


FIG. 4
THE LOAD OF CHLORIDES IN THE RIVER RHINE AT THE DUTCH - GERMAN BORDER

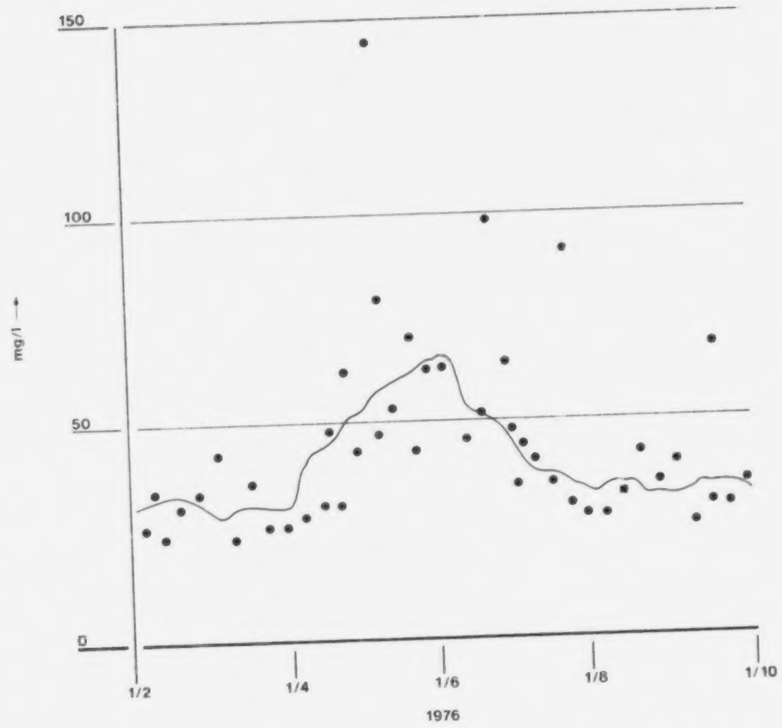


FIG. 5
COD-CONTANT OF SECONDARY EFFLUENT

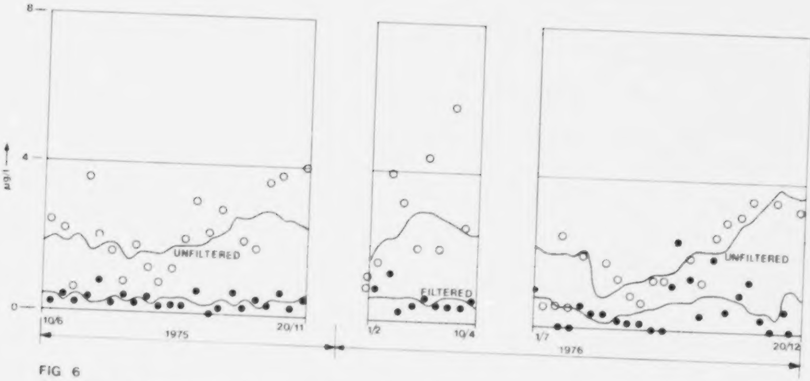


FIG. 6
CADMIUM CONCENTRATION IN SECONDARY EFFLUENT

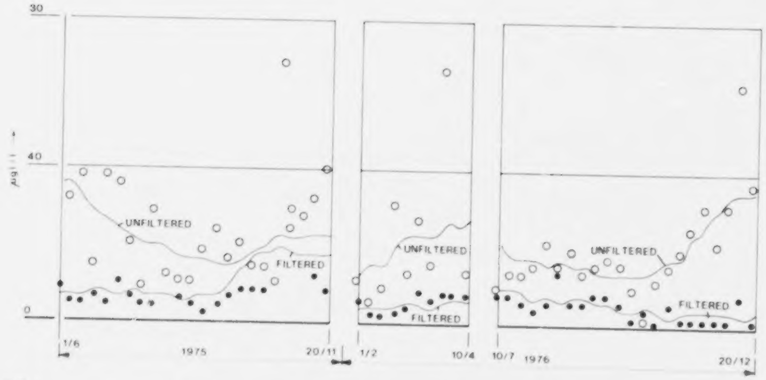


FIG. 7
LEAD CONCENTRATION IN SECONDARY EFFLUENT

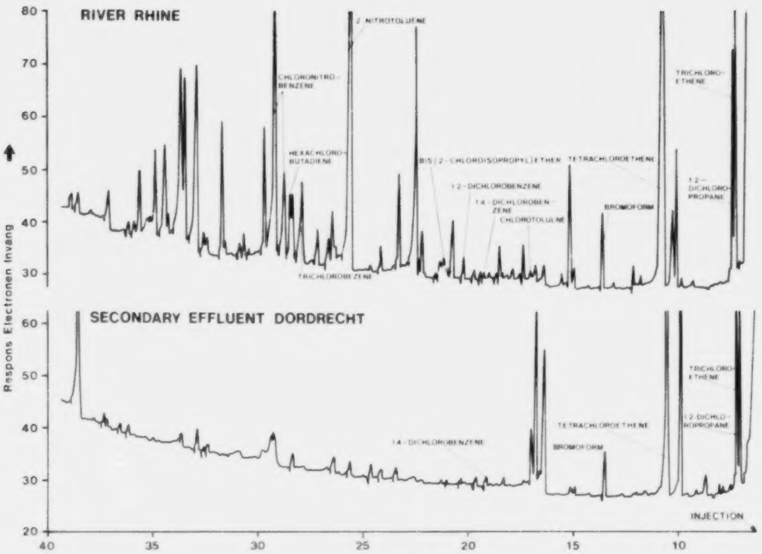


FIG 8
CHROMATOGRAM OF RIVER RHINE WATER AND SECONDARY EFFLUENT DORDRECHT

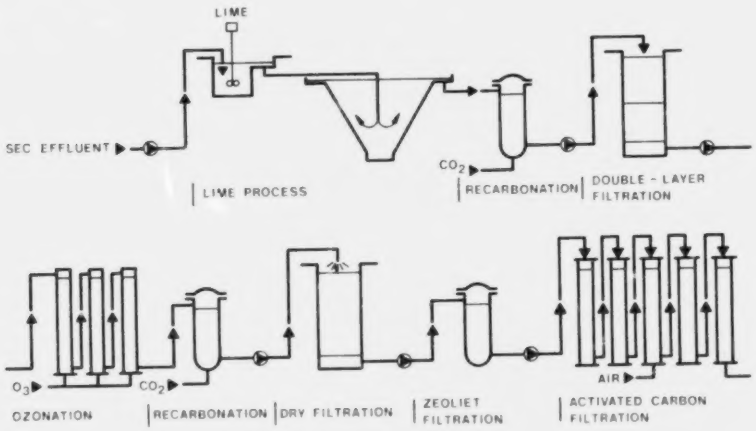


FIG 9
SCHEME OF THE FYSICAL CHEMICAL SYSTEM

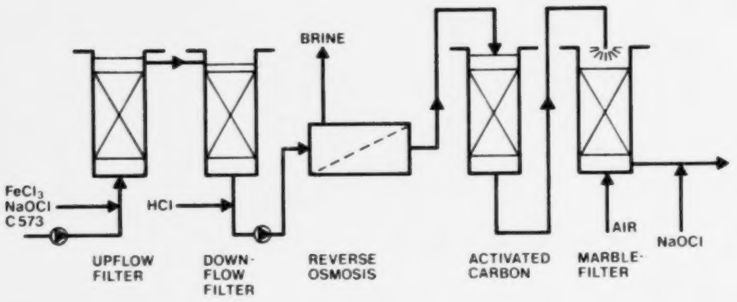


FIG. 10
SCHEME OF REVERSE-OSMOSIS SYSTEM

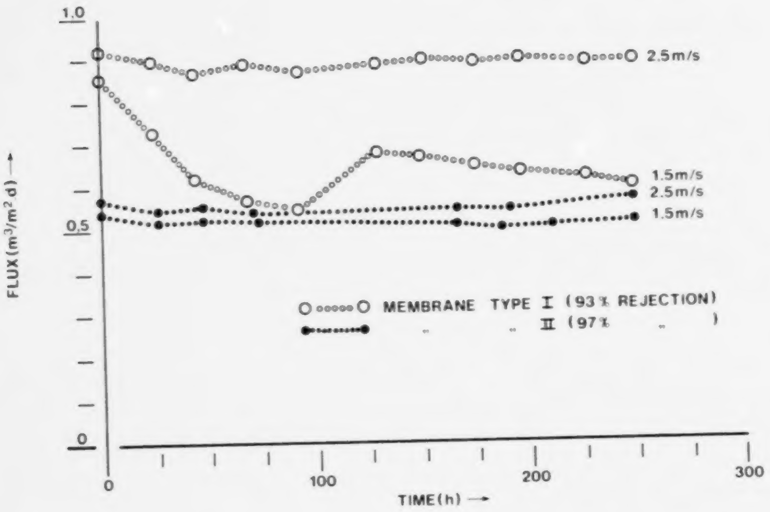


FIG. 11
FLUX V.S. VELOCITY ALONG MEMBRANES

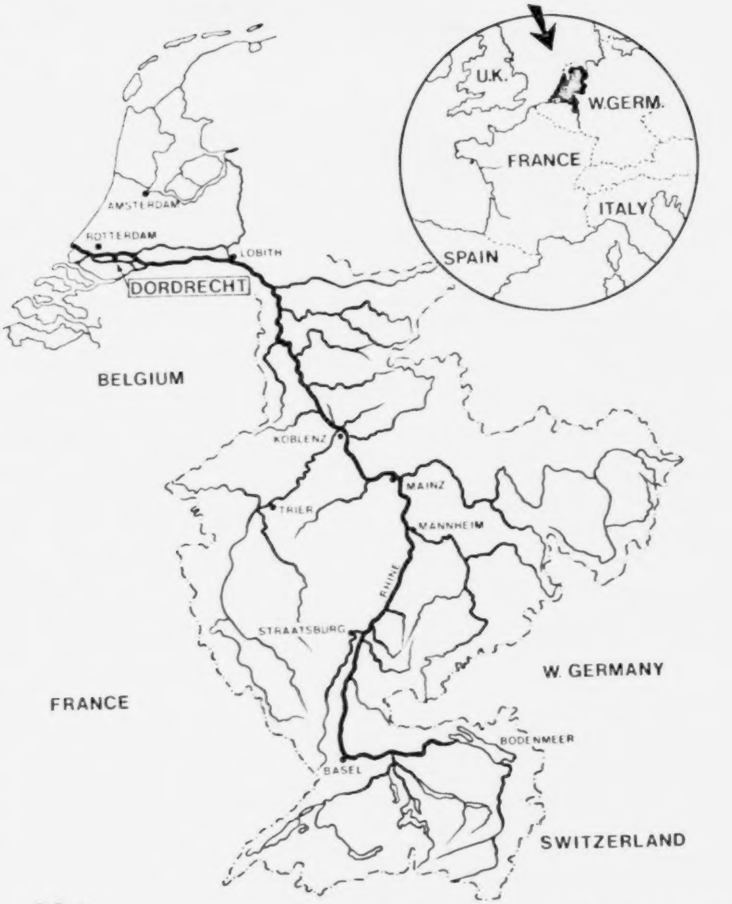


FIG. 1
HYDROLOGICAL MAP OF THE NETHERLANDS AND CATCHMENT AREA OF THE RIVER RHINE

DAN REGION PROJECT IN ISRAEL: FROM LABORATORY EXPERIMENTS TO FULL-SCALE WASTEWATER REUSE

E. Idelovitch, T. Roth, M. Michail, A. Cohen, R. Friedman
Sewage Reclamation Department
Tahal, Water Planning for Israel, Ltd.
P.O. Box 11170, Tel Aviv, Israel

A major problem that faces Israel's economy is the limited capacity of its available natural water resources, and, consequently, the difficulty to satisfy the increasing water needs of agricultural, industrial and municipal consumers. While desalination of sea water is supposedly the ultimate solution to this problem, reuse of municipal wastewater is regarded as an immediate priority, which is feasible both technically and economically. Wastewater treatment in Israel is, thus, carried out not only for pollution abatement but also for water supply augmentation.

The patterns of wastewater reuse cover a wide range – from surface storage of settled sewage or anaerobic oxidation ponds effluent for seasonal cotton irrigation, to advanced wastewater treatment and groundwater recharge (to potable aquifers) for unrestricted agriculture, industry and municipal non-potable uses. The reuse scheme adopted depends on objective, as well as subjective factors. The main objective factors are: the wastewater quantity and quality (community size and type) and the availability of potential users of treated wastewater. The subjective factors, which are numerous and sometimes overriding, include: the willingness of the users to negotiate with the water authorities exchange of fresh water for effluent, and the personal qualifications and inclinations of the engineers in charge of the water supply planning.

HISTORY OF DAN REGION PROJECT

The largest and most advanced wastewater reclamation scheme in Israel is the **Dan Region Project**, which provides for advanced treatment, groundwater recharge and indirect reuse of municipal wastewater from the largest urban area of the country – the Tel Aviv Metropolitan Area[1].

The project, which was first conceived more than a decade ago, came to fruition only in January 1977, when Stage I facilities (15 million cu.m/year or 11.5 mgd) were fully operated. The relatively long span of time which elapsed between the conception of the project and its operation is attributable to the complexity of the scheme, the novelty of some of the treatment processes adopted, and the difficulties involved in obtaining the institutional approvals required for its construction and operation.

The research work on the basic processes that were to be incorporated in the Dan Region Project started with laboratory experiments carried out in the 1960's at the Sanitary Engineering Laboratories of the Technion, Israel Institute of Technology, with wastewater from a residential suburb of the city of Haifa[2, 3]. In the 1970's extensive laboratory and pilot-scale studies were carried out by Tahal Ltd. and Mekorot Water Co. in the Dan Region Project itself, mainly on advanced treatment processes for reclamation of oxidation pond effluents. In the period 1974-1976, a large pilot and demonstration plant (350 cu.m/day or 100,000 gpd) for advanced wastewater treatment and groundwater recharge was operated continously, prior to the operation of the full-scale project.

The continuous progress of the research work and of the project planning led to considerable changes in the conceptual approach to municipal wastewater reuse, in the entire wastewater treatment pattern, and in some of the Stage I specific treatment processes. The major changes adopted relate to the reuse objectives and to the degree of treatment required (Figure 1).

The original idea of the project was to produce an effluent which can be reused indirectly (via groundwater recharge) for all purposes including domestic consumption. In 1975, a different approach was adopted – that of indirect non-potable reuse, i.e., reuse of the effluent (via groundwater recharge) for unrestricted agriculture, industries and municipal non-domestic uses. The adoption of this approach, which was dictated by a combination of several objective and subjective circumstances[4], implies conversion of the supply network in the south of the country to a "dual" system separately conveying potable water and high-quality non-potable water.

The original wastewater treatment pattern consisted exclusively of biological treatment in oxidation ponds, prior to groundwater recharge via spreading basins. The major changes adopted include: the addition to the Stage I Project (in 1970) of advanced treatment processes for upgrading the oxidation pond effluent as well as the adoption of a mechanical-biological treatment plant, based on a modified activated sludge process with nitrification-denitrification, for the Stage II Project. The latter will operate in parallel with Stage I facilities, the capacity of which will be increased to about 26 million cu.m/year or 20 mgd.

ADVANCED WASTEWATER TREATMENT

In Stage I of the Dan Region Project, which is now in full operation, wastewater from the Southern parts of the Tel Aviv Metropolitan Area undergoes three major treatment processes prior to groundwater recharge by means of spreading basins (Figure 2):

a. Biological treatment in recirculated oxidation ponds.
b. High lime-magnesium treatment in a sludge-blanket reactor-clarifier.
c. Detention in polishing ponds, for free ammonia stripping, natural recarbonation, and additional purification.

While the basic features of the treatment processes and their sequence have not changed since they were first proposed, some important modifications and additions have been developed and adopted in the full-scale plant operation, following pilot plant studies.

Three of the topics which were investigated at pilot and at full scale are reviewed in this paper:

(1) Low lime versus high lime magnesium treatment.
(2) Optimum pH and magnesium concentration for high lime treatment of the oxidation pond effluent.
(3) Recarbonation of the lime treated effluent.

Low Lime Versus High Lime-Magnesium Treatment

Lime treatment is the main process included in the Dan Region Project scheme (Stage I), as well as in many other reclamation projects. A distinction is usually made between the "low-lime process" carried out at a pH of 9 to 10, and the "high-lime process" at pH above 11. These two processes are different from the point of view of the functions they can perform and part of the chemistry involved, as well as the lime quantities and the coagulant aids they require. The most significant difference relates to the precipitation of magnesium as $Mg(OH)_2$, which occurs only during high-lime treatment, and enhances and diversifies the purification capacity of this process.

When the functions of lime treatment are limited to removal of phosphorus, residual suspended solids and some heavy metals which form relatively insoluble carbonates (e.g. lead and zinc), the low-lime process can be adopted. In this case the use of coagulant aids such as iron salts[5] or alum may serve, to a certain extent, as a substitute for $Mg(OH)_2$.

When the functions of lime treatment include removal of algae and color, preparation of the effluent for ammonia stripping, disinfection, removal of a variety of metals that form insoluble hydroxides (e.g. iron, copper, cadmium, manganese, lead, zinc, etc.), and removal of substances such as boron, silica and fluorides by adsorption on and co-precipitation with $Mg(OH)_2$, the high-lime process must be employed. Because of the importance of magnesium it has been suggested to refer to it as the "high lime-magnesium process"[6]. The most suitable coagulant aids in this process are magnesium salts (e.g. $MgCl_2$), the use of which is inevitable when the magnesium content of the wastewater is low.

Because of its capacity to remove algae, in addition to other pollutants, the high lime-magnesium process is most suitable for reclamation of oxidation pond effluents, especially when nitrogen removal is carried out by ammonia stripping, such as in the Dan Region Project.

The lime dose required for the high lime-magnesium process is much higher than for the low lime process, especially for high alkalinity water such as in Israel. A sharp rise in the lime doses required occurs particularly when pH is raised above the range 11.5-11.7 (Figure 3). However, if all the benefits accruing from high pH could be quantified, the economic return per unit of lime consumed might be higher in the high lime-magnesium process than in the low lime process.

Optimum pH and Magnesium Concentration for High-Lime Treatment

In the first laboratory experiments carried out in connection with this project, the optimum pH for clarification of oxidation pond effluent was found to be 11; the importance of the $Mg(OH)_2$ flocs formed in the process was recognized mainly with respect to their positive electrical charge, which neutralizes the negatively-charged algal colloids[2, 3]. However, the need to add magnesium ions from an external source was not determined, since the experiments were carried out with Haifa wastewater, which is richer in magnesium than that from the Tel Aviv area.

At later stages, the optimum pH and magnesium concentration were determined from jar tests and pilot plant studies carried out in the Project Area. It was found that the total magnesium concentration in the effluent should be about 50 mg/l in order to obtain optimum clarification results, and the optimum pH (at optimum magnesium concentration) in the range 11.3-11.5[7, 8, 9]. At the natural magnesium concentrations in the effluent (usually 10-25 mg/l), the clarification results can be improved by raising the pH to higher values, which imply higher lime requirements.

Figure 4 which is based on jar tests carried out in 1975 and 1977 shows the dramatic improvement in clarification of oxidation pond effluent which is obtained by addition of magnesium ions. The results obtained during the pilot plant operation in the period 1975-1976 demonstrated clearly the need to add magnesium from an external source[10,11]. The importance of magnesium addition to the optimum concentration was evident especially in winter, when the process efficiency is lower than in summer (Table 1 and Figure 5), mainly because of the higher solubility of the main reaction products – $CaCO_3$ and $Mg(OH)_2$. A commercial magnesium salt which is readily available in Israel and gave excellent results – $MgCl_2$ produced from the Dead Sea – was selected for the full-scale plant operation.

Recarbonation of Lime Treated Effluent

Along with its numerous merits, the high lime treatment process has one disadvantage: the pH must be reduced before the effluent can be reused or discharged to the environment. This is usually done by addition of carbon dioxide, which converts the hydroxides and carbonates to bicarbonates.

While the capacity of the polishing ponds to reduce the pH by absorption of CO_2 from the atmosphere and stripping of ammonia was determined in preliminary laboratory experiments[3], the original concept of the project included addition of CO_2 in the last polishing pond for completion of the recarbonation process, before pumping the effluent to the groundwater recharge basins. The studies carried out at pilot scale in the period 1975–76 showed that the first-stage recarbonation is almost completed in the polishing ponds at no extra cost. The alkalinity of the high lime effluent, which consists mainly of hydroxides, is reduced considerably by $CaCO_3$ precipitation in the polishing ponds. The residual alkalinity consists mainly of carbonates, which are converted only to a small extent to bicarbonates. The pH reached in the last polishing pond is usually in the range 9.5 –10.5, depending mainly on detention time and temperature (Figure 6). The natural recarbonation occurring in the polishing ponds is more efficient in winter than in summer, because of the better CO_2 absorption resulting from more favourable climatic conditions (temperature, wind, etc.). The studies also showed that the second-stage recarbonation can be carried out by addition of about 50 mg/l of CO_2 in order to convert all carbonates to bicarbonates and reduce the pH to 8-8.5[11,12].

However, the second-stage recarbonation (stabilization) process was not incorporated in the full-scale plant, since it was found that it occurs naturally during groundwater recharge. The upper soil layer of the spreading basins fulfils, among other functions, that of converting the carbonates (and residual hydroxides) to bicarbonates, presumably by means of CO_2 (and other acids) released by the soil bacterial population[13]. To ensure the completion of the first-stage recarbonation in the polishing ponds all year-round – in order to avoid scale deposition on the pumps and pipes which convey the water to the recharge basins – small amounts of neutral well water are pumped to the last polishing pond and blended with the effluent prior to groundwater recharge[14]. Such well water is readily available from several of the project recovery wells located in the vicinity of the oxidation and polishing ponds, which have been affected to a certain degree by effluent seeped from the ponds, and could not continue to pump potable water to the main supply network.

Recarbonation of the high lime effluent is, thus, carried out by a combination of three simple, low-cost processes (Figure 7):

- absorption of CO_2 from the atmosphere by long-time detention in the polishing ponds (first-stage recarbonation)
- blending with local well water (completion of first-stage recarbonation)
- percolation through the upper soil layer (second-stage recarbonation).

The addition of CO_2 which is expensive and requires certain sophistication is thus avoided.

SLUDGE TREATMENT

During the wastewater treatment processes incorporated in Stage I of the Dan Region Project, three types of sludge are produced, of which only one – the lime algae sludge from the reactor-clarifier – has to be removed and dealt with continuously. The other two are: the biological sludge which settles and undergoes biological degradation in the oxidation ponds, and the chemical sludge, consisting mainly of $CaCO_3$ produced during effluent carbonation, which settles in the polishing ponds.

The problems of dewatering, handling and disposal of the lime-algae sludge were not dealt with at all when the wastewater treatment scheme was devised, partly because of the difficulties involved in laboratory preparation of sludge and its study. During the pilot plant and the full-scale operations, the lime algae sludge was given particular attention. The studies carried out enabled the adoption of a "staged" approach, whereby an interim solution was implemented for the full-scale plant, and at the same time a research program was undertaken, aimed at sludge processing for materials recovery. This program is now in an advanced stage of pilot plant investigation.

The interim solution adopted for the sludge takes advantage of the prevailing climatic conditions in the project area, and makes use of natural sand drying beds for sludge dewatering and of natural lagoons for disposal of wet and dry sludge[15]. This solution, though simple and economic, is considered temporary since it is not acceptable, neither from an environmental point of view nor from the point of view of the plant operators, who found the operation of the drying beds difficult and the land available for lagoons limited[16].

The scheme developed for advanced sludge processing (Figure 8) which is now studied in a pilot plant incorporates (in order of priorities): separation of magnesium and its recycling to the process as coagulant aid in the form of $MgCO_3$ or $Mg(HCO_3)_2$, instead of $MgCl_2$ used at present; separation of organic matter rich in algal protein to be used as fertilizer or animal feed; and, eventually, lime recovery (with CO_2 as a by-product), if economic[17].

GROUNDWATER RECHARGE

Indirect reuse of municipal effluent by groundwater recharge, via spreading basins, should be preferred (where feasible) over direct reuse, because of the multiple functions fulfilled by the soil-aquifer system, which include: seasonal and multi-annual storage, additional purification, dilution with high-quality groundwater, and provision of a safety barrier. The success of groundwater recharge with municipal effluent depends on a series of factors, the most important of which are briefly discussed below.

Quality of Recharge Water. Wastewater must be treated prior to groundwater recharge in order to avoid pollution and minimize deterioration of the groundwater quality in the recharge zone. A suitable combination of biological and chemical treatment processes should be adequate for this purpose.

Recovery Wells. The recharge area must be surrounded by a sufficient number of adequately-spaced recovery wells, which can pump the bulk of the recharged water to uses compatible with its quality. The recharge-recovery operations should be balanced, after taking into consideration the storage capacity of the aquifer.

Infiltration Rates. Supervision and maintenance of the recharge basins should be regularly carried out so as to minimize soil clogging by physical, biological and chemical processes and, thus, to maintain high infiltration rates. Basin recharge can be carried out either by continuous flooding for long periods of time or by intermittent spreading, whereby wetting periods alternate with dry periods over a certain area. When flooding is continuous, the infiltration rate decreases sharply with time; when spreading is intermittent, infiltration rates are much better maintained (Figure 9). Experience gained from pilot experiments and from the full-scale recharge operation in the Dan Region Project in 1977-1978 showed that, in spite of "part-time" utilization of the area, the long-term hydraulic acceptance capacity of the basins is much higher for intermittent operation than for continuous flooding[11].

Groundwater Monitoring. A detailed hydro-geochemical monitoring of the aquifer in the recharge zone should be carried out in order to follow the movement of the recharged water in the aquifer, as well as the changes in the physico-chemical and biological quality of the groundwater.

Groundwater Quality

Vertical flow of the effluent through the well-aerated unsaturated zone, and horizontal flow in the saturated zone, provide for additional purification by a variety of biological, physical and chemical processes, such as filtration, biodegradation, nitrification-denitrification, adsorption, ion-exchange, oxidation, reduction, etc.

The changes in groundwater quality in any observation or recovery well in the recharge zone which results from these purification processes can best be described by means of three groups of quality parameters (Figure 10):

a. parameters such as **chlorides**, that are only affected by **dilution** and are not affected by the soil-aquifer purification capacity. The concentration of these parameters in the groundwater reaches levels similar to those in the recharge water, after a short period of time.

b. parameters that are affected mainly by **physico-chemical purification processes** in the soil-aquifer system, such as adsorption or ion-exchange (in addition to the dilution effect). The concentration of these parameters, such as **sodium** (cation exchange with Ca-Mg) rises in the groundwater much more slowly than that of chlorides; however, after a certain period (time lag), when the capacity of the soil is exhausted, their concentration reaches levels similar to those in the recharge effluent. The "time lag" depends mainly on the soil characteristics, which determine its capacity for adsorption and/or ion exchange, and on the distance from the recharge basin.

c. parameters that are affected mainly by **biological purification processes**, such as bacterial degradation of carbonaceous matter or nitrification-denitrification (in addition to the dilution effect). The concentration of these parameters such as **soluble COD**, reaches a certain peak during the period of acclimatization of the microbial population to the new conditions (concentration and composition of organic matter) and then decreases to lower levels, which presumably can be maintained, if the recharge basins are properly operated and the quality of the recharge water does not undergo abrupt changes.

EFFLUENT QUALITY AT VARIOUS TREATMENT STEPS

During the pilot plant operation in 1975-1976, a comprehensive evaluation of the effluent quality after each treatment step was made[10,11], for the two main seasons of the year – summer (20-30°C) and winter (10-20°C). As the operation of the full-scale plant in 1977-1978 was also accompanied by frequent analyses of the effluent after each treatment step, a comparison can be made between the major parameters characterizing effluent quality, as obtained in the pilot plant and in the full-scale plant (Table 2).

The main deviations of the full-scale plant results from those obtained in the pilot plant derive from physical and operational differences between the two plants, of which the most important are the following:

- Addition of $MgCl_2$ to the lime reactor clarifier in the full-scale plant was constant all year-round and limited to 10 mg/l (as Mg) for economic reasons, for simplicity of operation and in order to minimize chloride addition to an effluent which will eventually be used mainly for crop irrigation. Total Mg concentration thus varied between 20 and 40 mg/l, which is less than optimum.
- pH during high lime treatment was generally higher in the full-scale plant, especially in winter, in order to compensate at least partially for the insufficient magnesium addition.
- detention time in the full-scale polishing ponds was longer (at this stage) than in the pilot plant ponds.

Because of the poorer performance of the lime treatment process, on the one hand, and the longer detention time in the polishing ponds, on the other hand, the role of the latter in the full-scale plant was even more important than in the pilot plant. The functions performed by the polishing ponds included: free ammonia stripping, natural recarbonation, additional clarification, additional removal of organic matter, phosphorus and dissolved solids. The results obtained in the full-scale operation thus confirmed that the polishing ponds provide a most convenient unit operation following high lime-magnesium treatment[10].

The quality of the reclaimed water (after groundwater recharge and detention in the aquifer), which could not be determined in the pilot plant studies, has been monitored in the full-scale operation by means of observation wells located at various distances from the recharge basins (Table 2).

PROJECT IMPLEMENTATION AND OPERATION

The continuous production of a high-quality effluent for groundwater recharge, in a full-scale wastewater treatment plant, and the implementation of an extensive groundwater monitoring program in order to follow the effluent movement and quality changes in the aquifer, involve numerous technical, as well as administrative, difficulties. Furthermore, legal, psychological and public opinion problems arise from the vicinity of the treatment installations to urban areas and rural settlements, and from the proximity of the recharge area to private and municipal wells.

The success of a reuse project, which by its very nature, is both sophisticated and unpopular, depends to a great extent on the technical capacity of those in charge of its planning and construction, as well as on their ability to deal with institutional and public opposition, which are likely to accompany the project throughout its operation. More than in any other types of development projects, a reuse project must be based on specific, first-hand detailed data, which can supply the decision makers with the background information necessary for supporting the various phases of project planning, implementation and operation.

In the Dan Region Project, three major issues have been dealt with at all stages of project development and have given rise to much controversy in all phases of decision-making, namely:

- the degree of treatment to be provided prior to groundwater recharge,
- the need and desirability of recharging the effluent to a high-quality potable aquifer, and
- the conversion of the existing single supply network in the south of the country to a dual supply system, conveying separately potable water and high-quality non-potable water.

While most of the important decisions have already been made for Stage I of the project, which is now under operation, controversial proposals regarding each of the above mentioned topics are still under consideration for Stage II of the reclamation project, which is now in the planning stage. The data collected and the experience accumulated in the Stage I full-scale operation, in addition to that obtained from pilot plant studies, will undoubtedly play an important role in the decisions to be made on the configuration of the Dan Region Project – Israel's largest wastewater reclamation project – as well as of other, similar reclamation projects.

ACKNOWLEDGEMENTS

The Dan Region Project (Reclamation) has been sponsored by Mekorot Water Co. The authors would like to acknowledge the staff of Mekorot Water Co. who is in charge of the full-scale project operation and their colleagues in Tahal's Sewage Reclamation Department who were involved in the project design and pilot plant operation.

Authors Emanuel Idelovitch is Head of the Sewage Reclamation Department, Thea Roth is Senior Process Engineer, Medy Michail is Senior Chemical Engineer, Albert Cohen is Project Engineer and Robert Friedman is Senior Sanitary Engineer in the Sewage Reclamation Department, Tahal, Water Planning for Israel Ltd., Tel Aviv.

REFERENCES

1. IDELOVITCH, E., Wastewater Reuse by Biological-Chemical Treatment and Groundwater Recharge, *Jour. WPCF* (Dec. 1978).

2. FOLKMAN,Y., & WACHS, A., Reclamation of Stabilization Ponds Effluents Through Lime Treatment: Progress Report, Technion Research and Development Foundation, Haifa (in Hebrew) (1969).

3. FOLKMAN, Y., Reclamation of Stabilization Pond Effluent Through Optimal-Chemical Treatment, Ph.D. Thesis,TAHAL 022/71, Tel Aviv (in Hebrew) (1971).

4. IDELOVITCH, E., Wastewater Reclamation by Advanced Treatment, Dan Region Project – First Stage, Case Study, South Africa – Israel Symposium on Reuse of Wastewater in Agriculture and Industry, Tel Aviv (1975).

5. PARKER, D.S., et al., Lime Use in Wastewater Treatment: Design Cost and Data, U.S. Environmental Protection Agency, EPA – 600/2-75-038, (Oct. 1975).

6. IDELOVITCH, E., & MICHAIL, M., Trace Metal Removal from Secondary Effluent by Two-Stage High Lime-Magnesium Treatment, Paper presented at IAWPR 9th Conference (Stockholm), *Progress in Water Technology*, Vol. 10, Nos. 5/6, Pergamon Press (1978).

7. RONEN, M., & HALBERTHAL, J., Lime Treatment of Oxidation Pond Effluent, Progress Report, Mekoroth Water Co. (in Hebrew) (1975).

8. ROTH, T., Effects of pH and Magnesium Ion in Lime Treatment of Secondary Effluent, Paper presented at the IAWPR Workshop on Design-Operation Interaction at Large Wastewater Treatment Plants (Vienna) (1975).

9. MICHAIL, M. & IDELOVITCH, E., Research on Magnesium Addition in High Lime Treatment of Oxidation Pond Effluent, Progress Report, TAHAL – Water Planning for Israel Ltd., Sewage Reclamation Department, Tel Aviv (in Hebrew) (1975).

10. IDELOVITCH, E., et al. Advanced Treatment and Reuse of Municipal Wastewater, Dan Region Pilot Plant, Summer Operation, TAHAL - Water Planning for Israel, Sewage Reclamation Department, Publication No. 01/76/38, Tel Aviv (1976).

11. IDELOVITCH, E., et al., Advanced Treatment, Groundwater Recharge and Reuse of Municipal Wastewater, Dan Region Pilot Plant, Winter Operation, TAHAL - Water Planning for Israel Ltd., Sewage Reclamation Department, Publication No. 01/77/40, Tel Aviv (1977).

12. MICHAIL, M., Research on Two-Stage Recarbonation of High-Lime Effluent by Liquid Carbon Dioxide. Progress Report. TAHAL - Water Planning for Israel Ltd., Sewage Reclamation Department. Tel Aviv (in Hebrew) (1974).

13. IDELOVITCH, E., et al., Groundwater Recharge with Municipal Effluent. Dan Region Sewage Reclamation Project. Annual Report. TAHAL - Water Planning for Israel., Sewage Reclamation Department, Publication no. 01/78/31. Tel Aviv (1977).

14. MICHAIL, M., & IDELOVITCH, E., Research on Tertiary Effluent Stabilization by Blending with Well Water. Progress Report. TAHAL - Water Planning for Israel Ltd. Sewage Reclamation Department, Tel Aviv (in Hebrew) (1975).

15. COHEN, A., & IDELOVITCH, E., Properties and Treatment of Lime-Algae Sludge. Paper presented at IAWPR 9th Conference (Stockholm). *Progress in Water Technology*, Vol. 10, Nos. 5/6, Pergamon Press (1978).

16. MARCOVICI, M., & KENRAK, A., Mekorot Water Co., Dan Region Unit, Personal Communication (1977).

17. IDELOVITCH, E., et al., Recovery of Water, Chemicals and Animal Feed from Municipal Wastewater. Paper presented at the 8th Scientific Congress of the Israeli Ecological Society, Tel Aviv (1977).

TABLE 1

Effect of Mg Concentration on Clarification of Oxidation Pond Effluent by High Lime Treatment

Winter Operation[10]

	Total Mg Concentration	
	*40 mg/l**	*50 mg/l**
Clarification pH	11.4	11.4
Suspended Solids, *mg/l*		
Secondary Effluent (SE)	240	225
High Lime Treatment (HLE)	98	57
Tertiary Effluent (TE)	22	9
Relative Removal Efficiency, %		
High Lime-Mg Treatment	60	75
High Lime-Mg & Polishing Ponds	91	96

Summer Operation[11]

	Total Mg Concentration	
	*30 mg/l***	*50 mg/l**
Clarification pH	11.6	11.4
Suspended Solids, *mg/l*		
Secondary Effluent (SE)	175	110
High Lime Effluent (HLE)	45	25
Tertiary Effluent (TE)	18	11
Relative Removal Efficiency, %		
High Lime-Mg Treatment	75	77
High Lime-Mg & Polishing Ponds	90	90

* Consisting of natural Mg concentration in secondary effluent (10-25 mg/l) plus $MgCl_2$ addition

** Without Mg addition

Note: Based on pilot plant results

TABLE 2

Effluent Quality at Various Treatment Steps

Pilot Plant

Parameter	Secondary Effluent		High Lime Effluent		Tertiary Effluent	
	Summer	Winter	Summer	Winter	Summer	Winter
pH	8.3	8.2	11.5	11.5	10.6	10.0
Suspended Solids	130	225	30	77	10	13
BOD	45	57	8	12	3	3
COD	320	350	100	113	85	63
Ammonia	27	36	20	30	3	10
Total Nitrogen	39	51	26	36	6	13
Phosphorus	10.7	11.8	1.0	3.0	0.2	1.5
Dissolved Solids	870	770	690	650	630	660

Full Scale Plant

Parameter	Raw Sewage		Secondary Effluent		High Lime Effluent		Tertiary Effluent		Reclaimed Water
	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	
pH	7.9	7.9	8.3	8.3	11.7	12.1	10.1	10.2	8.0
Suspended Solids	230	249	212	247	55	154	8	20	0
BOD	230	226	60	53	13	19	7	6	0.2
COD	510	465	355	298	90	125	75	67	10
Ammonia	38	46	21	27	24	30	1	5.5	0.02
Total Nitrogen	58	70	53	55	31	39	6	10.5	5
Phosphorus	11.9	12.0	11.6	11	2.0	1.6	0.5	0.8	0.05
Dissolved Solids	875	778	860	739	830	737	640	588	630

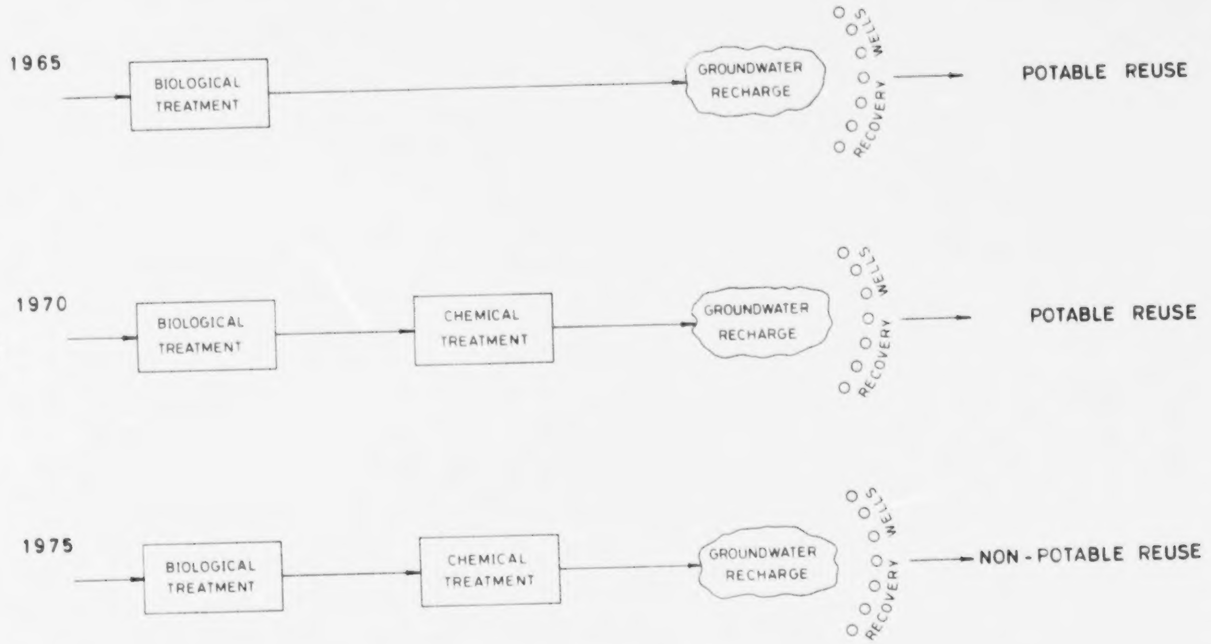


Fig. 1. Evolution of Dan Region Reclamation Scheme

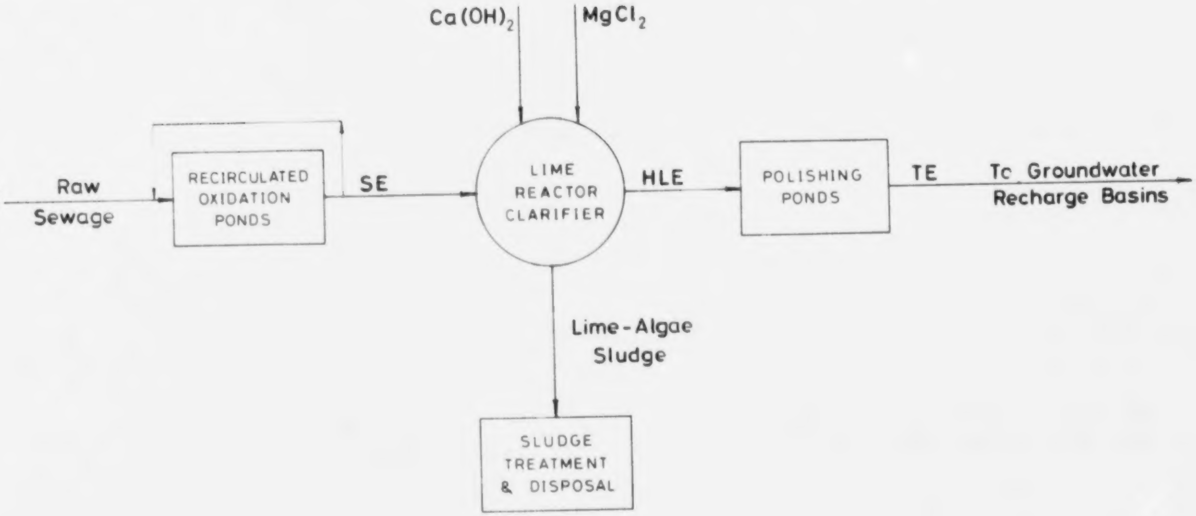
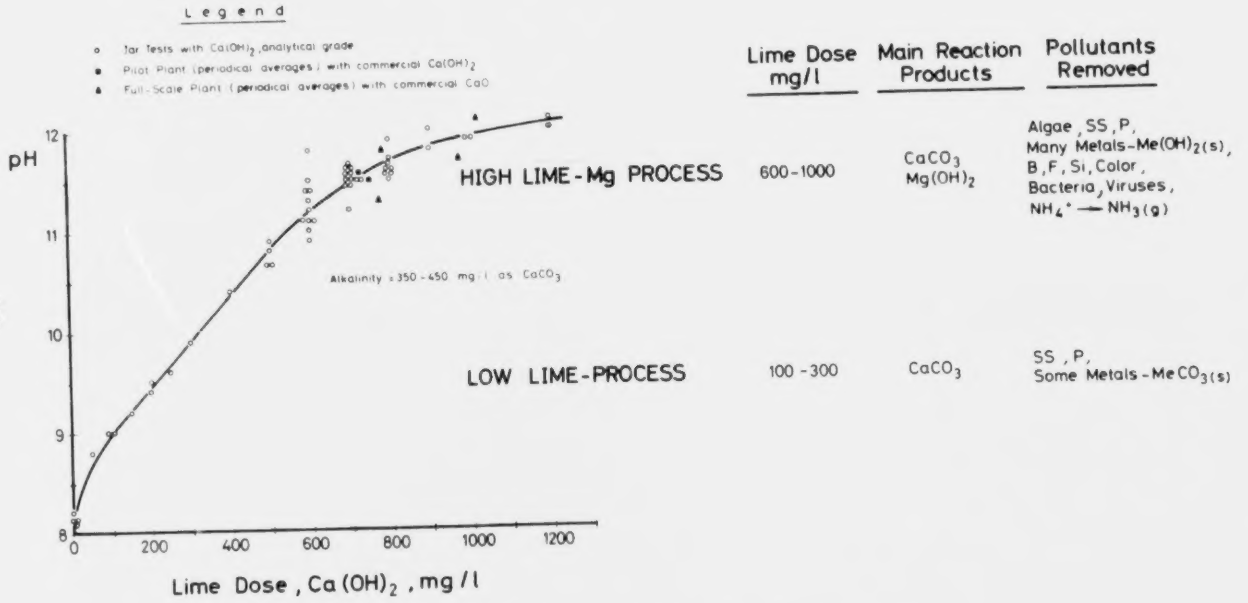


Fig. 2. Dan Region Sewage Reclamation Project: Stage I Flow Diagram



	Lime Dose mg/l	Main Reaction Products	Pollutants Removed
HIGH LIME-Mg PROCESS	600-1000	$CaCO_3$ $Mg(OH)_2$	Algae, SS, P, Many Metals-$Me(OH)_{2(s)}$, B, F, Si, Color, Bacteria, Viruses, $NH_4^+ \longrightarrow NH_{3(g)}$
LOW LIME-PROCESS	100 -300	$CaCO_3$	SS, P, Some Metals-$MeCO_{3(s)}$

Fig. 3. Low Lime vs High Lime-Magnesium Process

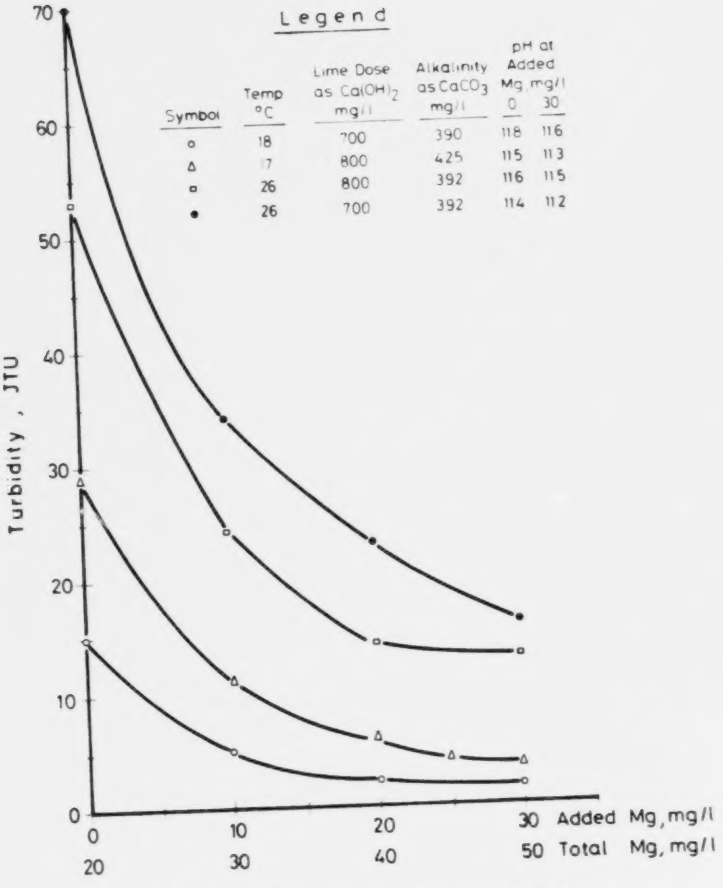


Fig. 4. Effect of Mg Addition on Clarification Results, Jar Tests

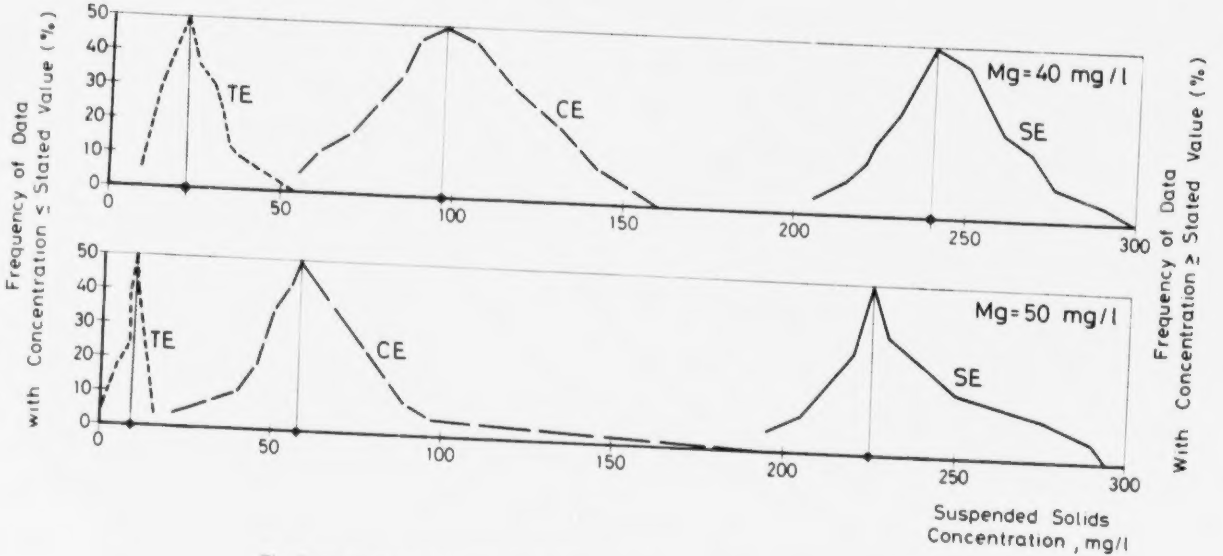


Fig. 5. Effect of Mg Addition on Clarification Results: Pilot Plant, Winter Operation

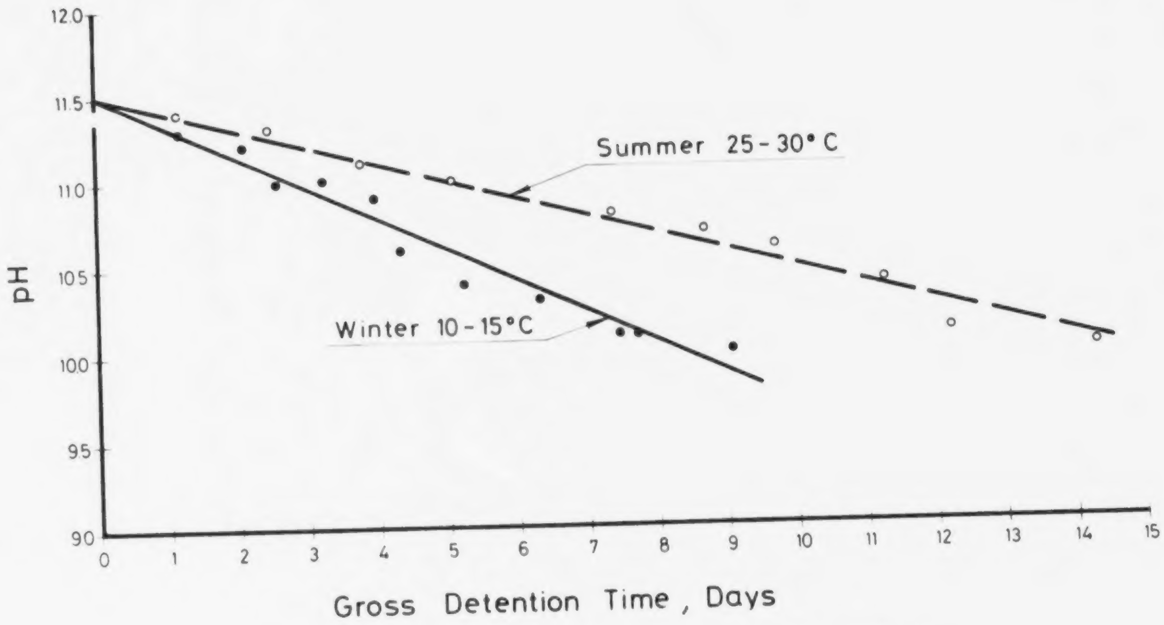


Fig. 6. pH Decrease by Natural Recarbonation in Polishing Ponds; Pilot Plant

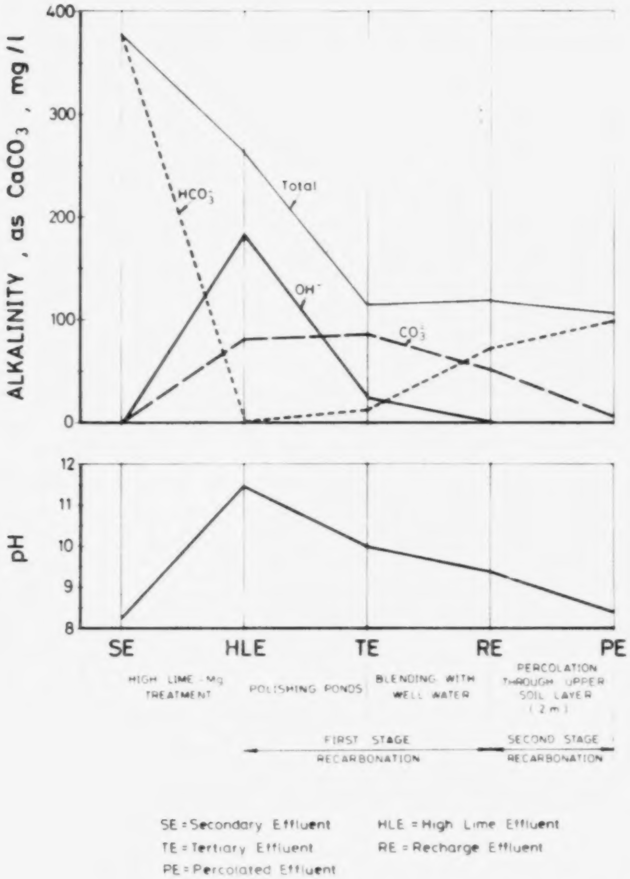


Fig. 7. Changes of pH and Alkalinity Forms throughout the Treatment Scheme

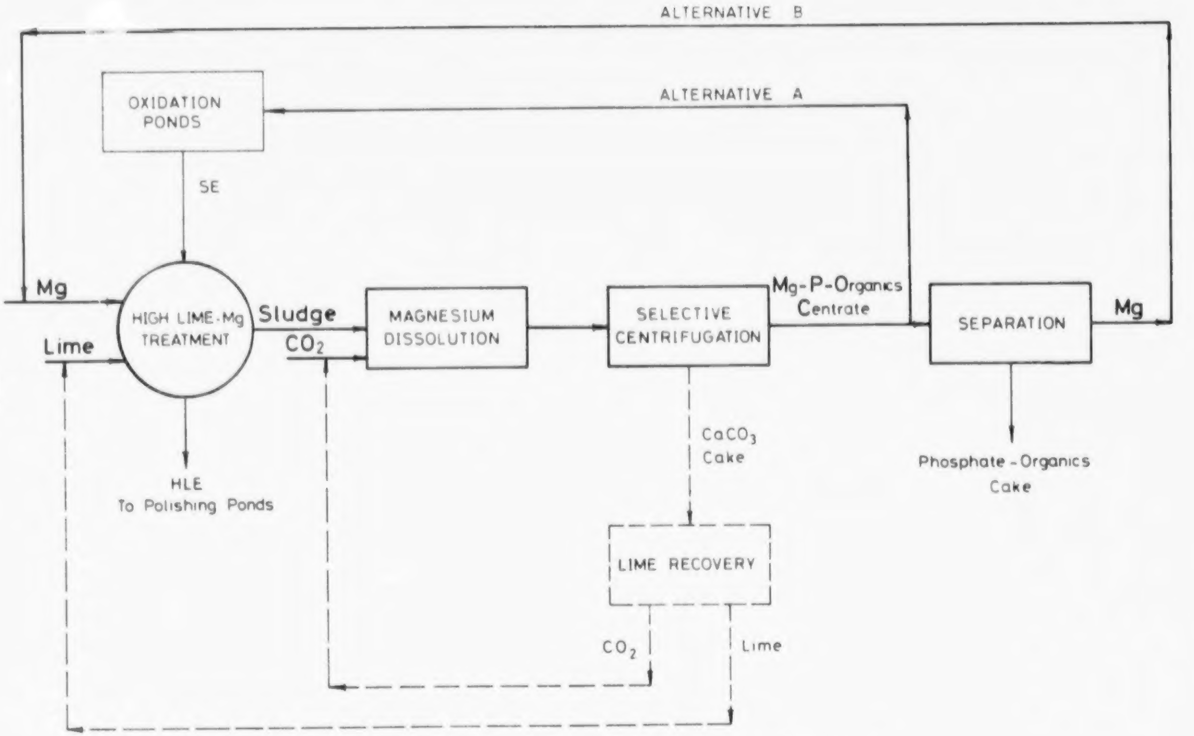


Fig. 8. Lime Algae Sludge Processing for Recovery of Chemicals

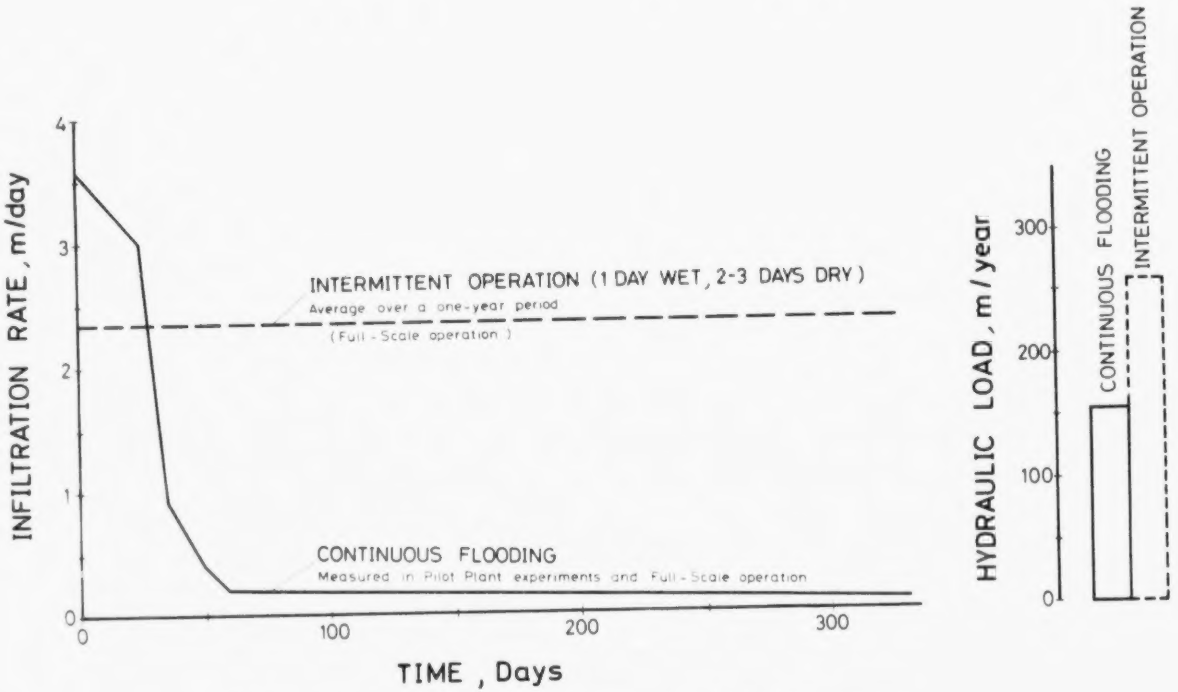


Fig. 9. Infiltration Rate and Hydraulic Load for Continuous and Intermittent Flooding

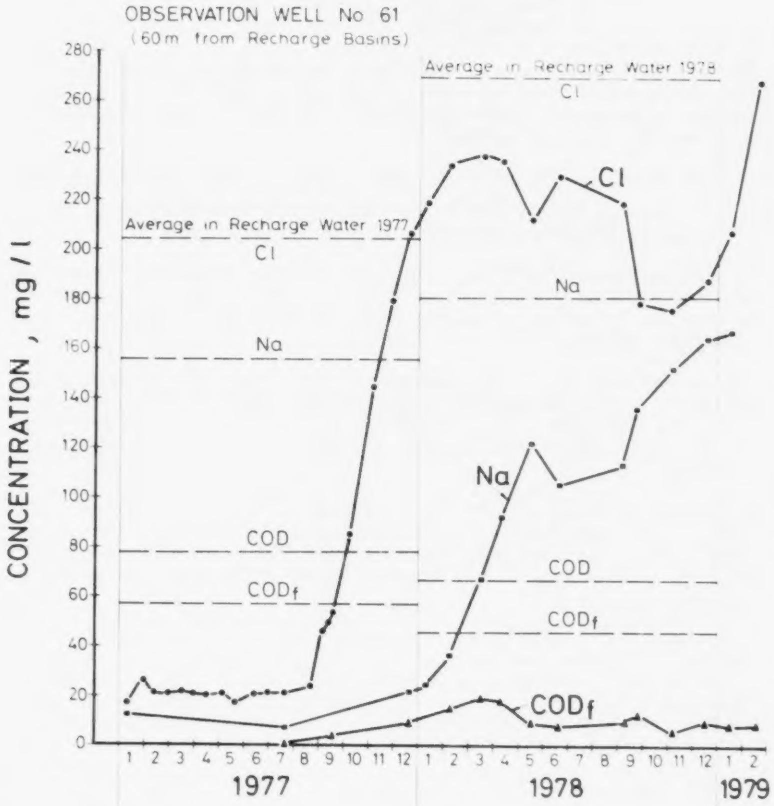


Fig. 10. Effect of Recharge on Groundwater Quality Changes

THE DAN REGION WASTEWATER PROJECT - STAGE II: AN ADVANCED SINGLE STAGE BIOLOGICAL WASTEWATER TREATMENT AND RENOVATION SYSTEM

David J. Farchill
Project Engineer
Tahal Consulting Engineers Ltd.
P.O.Box 11170, Tel Aviv, Israel.

The Dan Region Wastewater Project is a comprehensive inter-regional scheme for the collection, purification and reclamation of the municipal wastewater generated within the Greater Tel Aviv Metropolitan Area with two principal objectives:

1. To eliminate current pollution along the Tel Aviv shore line; and
2. To recycle the reclaimed water to the arid region in Southern Israel, mainly for industrial and agricultural uses.

Israel, at present, utilizes practically all its fresh water potential of 1,500 million cu.m/year (1,155 mgd.) Of this quantity, approximately 75% is supplied to agriculture, and 25% used for domestic and industrial purposes.

The anticipated reclamation capacity of the project by the year 1985 (Stage II), with all potential contributors connected to the system, will produce a high quality irrigation and industrial water source equivalent to 10% of the total current agricultural water used in the country.

This sizable amount of water has been included in the overall national water budget, and will permit the transfer of an equivalent amount of fresh water to the domestic sector, to meet the rapidly growing urban demands, until more expensive water sources can be economically developed.

The Scope of the Stage II Project is to collect all the municipal waste waters from the Dan Region, and to renovate the effluent to the highest possible level achievable by an advanced biological treatment process. The final reclamation, storage and reuse of the renovated effluent produced by the Stage II facilities, are part of separate projects currently in the advanced stages of planning and design.

EXISTING CONDITIONS

The Dan Region contains the largest city in Israel and six smaller towns, which together constitute one large continuous urban area. In 1956, the seven municipalities, including Tel Aviv-Yafo, Ramat Gan, Bnei Beraq, Givatayim, Petah Tiqwa, Holon, and Bat Yam, organized the Dan Region Association of Towns (Sewage), as a statutory authority to deal in all matters concerning municipal wastewater within the boundaries of the Association. It is anticipated that within the near future, two additional municipalities (Rishon-Le-Zion and Rehovot) will join, or participate in the Association's current expansion program.

Currently, only a portion of the wastewater from the municipalities of Holon, Bat Yam and the southern section of Tel Aviv-Yafo, (tributary to the Bassa Pump Station), are conveyed to existing oxidation pond treatment facilities, located 10 km south of the City of Tel Aviv-Yafo

(Soreq Treatment Plant Site). Prior to final disposal by ground water recharge, the effluent from the ponds receives lime treatment for removal of algae and ammonia, as a part of the Project's Stage I program.

The remainder of the wastewater from the Dan Region Association of Towns service area (tributary to the Reading Pump Station), encompassing approximately two thirds of the total regional flow, is discharged to sea through the existing outfall facilities at the Reading Station. Wastewater screening facilities have been recently constructed at this location, as part of an interim program, to alleviate current pollution conditions along the adjoining sea shore, until the completion of the Stage II Project.

WASTEWATER QUALITY

As a part of the planning activities for the transmission and treatment systems, a comprehensive wastewater survey was undertaken at the principal collection points of the system. Table No. 1, indicates representative Dan Region wastewater quality values, as sampled at the Reading Pump Station during successive years. Roughly, 90% of the total wastewater is domestic sanitary sewage; the remaining 10% is derived from industrial sources. Storm water flows are collected and disposed of through a separate drainage network.

The wastewater year in the Dan Region is equally divided into two distinctive seasons. The period termed "High Load" falls during the winter and spring seasons, when lower sewage temperatures prevail (December - May); the period termed "Low Load" covers the summer and fall seasons, when higher sewage temperatures prevail (June - November). The higher organic loading during the winter season reflects the heavy concentration of citrus processing industries in the Dan Region. A review of Table No. 1, further indicates a considerable reduction in the organic load (BOD/COD) since 1976, due to a very active policy of industrial waste control undertaken by the Dan Region Association of Towns in the past few years. This has had a direct influence in reducing the design capacity of the proposed treatment facilities and in the reduction of residual trace pollutants in the effluent.

PLANNING CONSIDERATIONS

Upon completion of the Stage II Project, the raw wastewater will be collected and conveyed to a central regional treatment facility located adjacent to the existing oxidation ponds. The two combined facilities will ultimately serve the nine above mentioned municipalities, encompassing an urban service area of 200 sq.km, with an estimated (1977) population of 1.1 million residents, and including approximately 800 industrial and commercial enterprises generating a wide range of water borne industrial wastes. The overall system has been planned for a projected expansion to 1.3 million residents by 1985 (Stage II) and 1.7 million by the year 2000.

The expansion of the treatment facilities and of the main transmission system is currently under construction, involving a total investment of over US$ 65 million, through the completion of the 1985 stage of development. The complete transmission system, and the first phase of the biological treatment plant construction, which will be placed in operation in 1980-81, are financed by the joint participation of the local municipalities, the State of Israel, and the World Bank.

The wastewater transmission system, 18 km long, comprises sections of force mains and gravity flow lines up to 84 inches in diameter; the two major existing sewage pump stations (Reading and Bassa) will lift the raw sewage a total of 40 metres, at a maximum rate of 25,000 cu.m per hour, with pumping equipment rated at 1,600 H.P. and 8,800 H.P. respectively, at each of the two facilities, after completion of the current expansion program.

The proposed regional wastewater reclamation facilities will have an average treatment capacity of 116 million cu.m per year (84 U.S. MGD) by the year 1985 (Stage II), with provision for expansion to treat 160 million cu.m per year (116 U.S. MGD), after the year 2000, when the population of the Dan Region is projected at 1.7 million. The existing oxidation ponds will continue to operate in parallel with the new facilities, and will effectively provide hydraulic and organic peak loading equilization for a more efficient operation of the proposed biological treatment plant.

During the course of planning the Stage II Project, several alternative plant locations and process strategies were investigated for technical, economic and environmental feasibility. The problem of plant location focussed primarily on the alternatives of several local plants, dispersed throughout the Dan Region, as against one single regional facility at the Soreq Site. Cost, land availability, and operational flexibility factors, complemented by a comprehensive environmental impact evaluation of the overall project , fully justified the selection of a centralized regional treatment strategy.

The question of treatment process selection, centered primarily on the consideration that the effluent should be suitable for unrestricted agricultural use if recharged into a controlled aquifer for seasonal storage equilization and maturation; additional polishing and disinfection would be provided to the secondary effluent, as required, before final reuse. Because of the possibility that the reclaimed effluent may ultimately reach active water supply aquifers, a denitrified effluent with a maximum of 10 mg/l of Nitrate-N, and a high level of dissolved COD removal, were established as part of the water quality criteria for the Stage II Project. Alternative wastewater renovation strategies considered,included physical-chemical treatment and high-rate activated sludge, all followed by ammonia stripping. In view of the high soluble BOD and organic nitrogen concentrations in the Dan Region wastewater, the above alternatives were ruled out, due to either poor carbon removal efficiencies, uncertainty of process stability, or the high costs inherent in the multiple stages of the various process trains.

The process ultimately selected for the Stage II Project will include, initially, secondary biological treatment and chlorination, with provisions for additional polishing and/or physical-chemical treatment, if required in the future. The biological treatment will consist of a modified, low-rate, activated sludge process, without primary settling, which will provide for efficient carbonaceous BOD/COD removal, nitrification and denitrification, and a high level of phosphorous removal, in a single sludge system, without exogenous carbon sources or chemical additions. The modified activated sludge process was developed during the past five years, especially for the renovation of the combined domestic and industrial wastewater prevailing in the Dan Region. During the past three years the process has undergone comprehensive technical scale pilot tests at the Reading Demonstration Plant and Laboratory Facilities in North Tel Aviv. The pilot plant performance is continuously monitored by computerized statistical methods, to provide detailed criteria for the planning of the full scale plant, which is currently in the advanced stages of design.

DEMONSTRATION PLANT RESULTS

The operational program of the demonstration plant, generally follows the seasonal pattern of wastewater quality indicated in Table No. 1. To minimise the manpower requirements, the plant is fully automated for 24-hour mechanical operation and process monitoring. The facility consists principally of two identical activated sludge process trains with a variable inflow capacity of up to 2.5 cu.m/hour. At the prevailing organic concentrations, each process train has an equivalent treatment capacity of approximately 1,500 P.E. (Population Equivalent), when operated at the lower organic loading levels.

In 1974, during the early planning stages of the Stage II Project, the following average values were established for the principal biological effluent quality criteria to be met by the proposed modification of the activated sludge process:

BOD_5	12 mg/l
BOD_5 - Filt.	6 mg/l
S.S. - 105 °C	10 mg/l
Total Nitrogen	8 mg/l
Ammonia - N	7 mg/l
Nitrate - N	3 mg/l

Tables No. 2 and No. 3, respectively, indicate the performance results of the demonstration plant during two subsequent (1976-77 and 1978) operating periods, including both winter and summer operation. The previously stated effluent quality criteria have been fully met, during both periods reported herein, and have been further improved during the current operating season. The process has shown a remarkable degree of operational stability throughout the full range of temperatures and wastewater quality variations prevailing in the Dan Region, and has exhibited a very high efficiency in removing BOD and COD as well as Nitrogen and Phosphorous, without the addition of exogenous carbon sources or chemicals. The excellent nutrient removal capacity exhibited by this system, is an important asset in the renovation process and ultimate reuse of this valuable water resource.

ACKNOWLEDGEMENTS

The Dan Region Wastewater Project - Stage II is sponsored jointly by the Dan Region Association of Towns (Sewerage), Mr. Y. Caspi, Chairman; and the Israel Sewerage Project, a national water pollution control program, under the direction of Mr. M. Ben Meir, State of Israel Water Commissioner.

The author would like to acknowledge the staff of the Dan Region Wastewater Project Division, Tahal Consulting Engineers Ltd., and in particular Mr. Y. Gruber, Project Manager, Mr. M. Goldstein, Senior Civil Engineer, and Mr. C. Arueste, Sanitary Engineer for their valuable assistance in the preparation of this paper.

TABLE 1

Dan Region Wastewater Quality

Parameter (mg/1)	Year	High Load Season (December-May)		Low Load Season (June-November)	
		Full Workdays	Sat. and Holidays	Full Workdays	Sat. and Holidays
BOD_5	1976	475	-	357	236
	1977	396	234	326	202
	1978	362	233	346	217
COD	1976	1,080	-	783	556
	1977	730	536	524	388
	1978	688	477	665	455
SS-105 °C	1976	355	-	299	274
	1977	238	261	236	194
	1978	308	224	332	252
NKT	1976	68.8	-	58.6	66.7
	1977	57.3	65.6	49.4	51.2
	1978	51.2	57.4	46.3	48.5
P Total	1976	-	-	13.9	12.4
	1977	11.6	12.7	13.2	14.2
	1978	11.6	11.8	14.5	13.5
CHLORIDE	1976	334	-	341	263
	1977	315	244	334	254
	1978	315	256	329	269
ALKALINITY	1976	334	-	341	263
	1977	315	244	334	254
	1978	315	256	329	269

TABLE 2

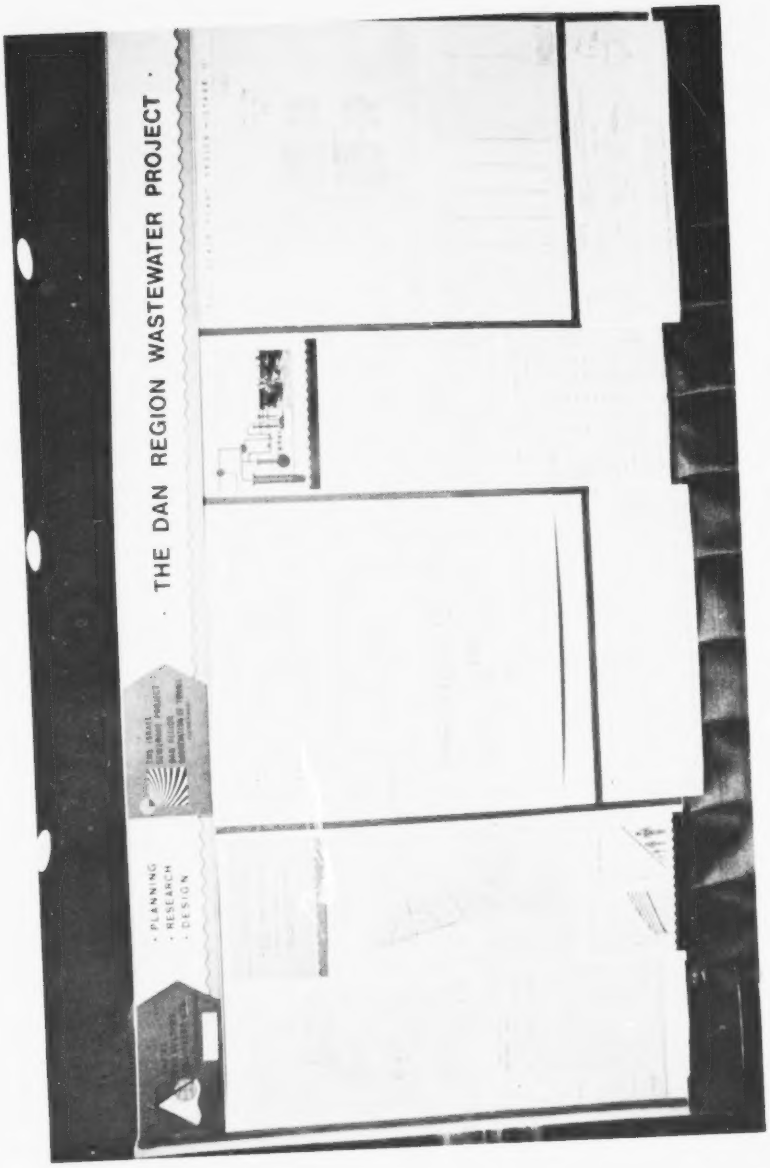
Reading Demonstration Plant Facilities
Quality of Influent, Effluent and Removal Efficiency
1976-77 Pilot Operation

Parameter (mg/1)	Summer Season Low-Load (18 Weeks)			Winter Season High-Load (16 Weeks)		
	Infl.	Effl.	% Rem.	Infl.	Effl.	% Rem.
BOD_5	325	5.7	98.2	351	5.8	98.3
BOD_5 - Filt.	162	3.9	97.6	189	3.4	98.2
COD	718	57.6	92.0	747	66.6	91.0
COD - Filt.	284	44.1	84.5	329	49.8	84.9
S.S.- 105 °C	303	9.9	-	301	9.2	-
TOTAL NITROGEN	59.0	8.2	86.5	57.5	9.1	84.0
AMMONIA - N	49.2	1.1	97.8	47.3	1.7	96.4
NITRATE - N	Nil	6.2	-	Nil	6.9	-
PHOSPHOROUS	14.2	5.3	62.3	12.6	2.8	78.2
ALKALINITY ($CaCO_3$)	458	314	30.8	421	293	30.3
pH	-	8.1	-	7.6	8.0	-
LIQUID TEMP. °C	-	26.0	-	-	17.2	-

TABLE 3

Reading Demonstration Plant Facilities
Quality of Influent, Effluent and Removal Efficiency
1978 Pilot Operation

Parameter (mg/1)	Summer Season Low-Load (15 Weeks)			Winter Season High-Load (16 Weeks)		
	Infl.	Effl.	% Rem	Infl.	Effl.	% Rem.
BOD_5	303	N/A	-	357	N/A	-
BOD_5 - Filt.	133	4.4	96.7	171	4.3	97.5
COD	647	62.1	90.4	716	68.4	90.4
COD - Filt.	205	44.4	78.3	263	49.4	81.2
S.S.- 105 °C	322	11.5	96.4	319	9.6	97.0
TOTAL NITROGEN	44.9	3.0	93.3	53.6	3.1	94.2
AMMONIA - N	37.0	1.1	97.0	41.8	0.9	97.8
NITRATE - N	Nil	1.0	-	Nil	1.2	-
PHOSPHOROUS	14.2	3.2	77.5	12.6	3.7	70.6
ALKALINITY ($CaCO_3$)	399	309	22.6	409	311	24.0
pH	7.9	8.2	-	8.0	8.1	-
LIQUID TEMP. °C	-	29.2	-	-	23.3	-


· THE DAN REGION WASTEWATER PROJECT ·
· PLANNING
· RESEARCH
· DESIGN

DEVELOPMENT OF TWO SEWAGE AND SOLID WASTE TREATMENT SYSTEMS WITH POTENTIAL FOR WATER REUSE AND ENERGY RECOVERY

Duncan K. Smith
Assistant Director, Applied Chemistry
Ontario Research Foundation

Richard V. Laughton
Associate Research Scientist, Applied Chemistry
Ontario Research Foundation

INTRODUCTION

The Ontario Research Foundation, a not-for-profit research and development organization formed by the Ontario Government and Canadian manufacturers in 1928, has been engaged in sponsored research on water reuse projects since 1957. Between 1957 and 1965 a number of self-contained household sewage disposal units were developed and prototype units with recycling of the effluent for toilet flushing purposes were installed and tested in research houses built by the National House-builders Association. A much larger self-contained sewage disposal system with reuse of treated effluent for toilet flushing was also designed and installed in a 200 pupil school on Baffin Island in the Canadian North. Water savings of the order of 40% were achieved with fairly simple activated sludge systems treating sanitary sewage or sanitary sewage plus a small amount of grey water only. Later work from 1965 onwards was directed toward more ambitious systems in which all sewage wastes and garbage of domestic origin were treated. One system, named CANWEL (Canadian Water Energy Loop), has been developed for the Federal Government's housing agency - Central Mortgage and Housing Corporation (CMHC) in which all domestic wastes from an apartment complex are treated on site with reclamation of heat and the potential for reuse of the highly treated waste-water. The other major system under development for the Defence and Civil Institute of Environmental Medicine (DCIEM), is a transportable Environmental Service Module (ESM) for military or civilian use in remote northern settlements.

DEVELOPMENT OF AN ENVIRONMENTAL SERVICE MODULE (ESM)

Design Alternatives

The Ontario Research Foundation (ORF) under contract to the Defence and Civil Institute of Environmental Medicine (DCIEM) developed the conceptual design of the Environmental Service Module (ESM). The ESM is an integrated facility for the regeneration of water of potable quality from human waste. Both liquid and solid human wastes are handled by the unit.

Objectives set out for the ESM specified that the unit should employ as much water recycling as practically possible, be transportable in Hercules C-130 aircraft, contain complete consumer facilities such as toilets, showers and laundry area, and produce an innocuous, sterile, compact residue which could be easily and safely disposed of.[1] Extensive water recycling was to be employed for all purposes except drinking and culinary use.[2]

A major portion of the study conducted by ORF was devoted to choosing the waste treatment system for the ESM. The waste treatment unit forms the technical basis for the whole ESM concept.[3] The three alternatives evaluated by ORF were:

(1) biological treatment with incineration of sludges and municipal waste,

(2) physical-chemical treatment with incineration of sludges and municipal waste, and

(3) wet air oxidation of both sanitary and municipal waste.

An exhaustive comparison of the alternatives led to a recommendation to use wet air oxidation as the basis for the complete treatment process system. The use of wet oxidation in modular treatment systems had previously been studied by the Office of Research and Development of the U.S. Coastguard[4] and the U.S. Navy.[5]

The design specifications selected for the conceptual design were obtained from the literature and past experience of ORF staff members in modular treatment systems. A population size of 100 people was selected; however, populations as low as 50 or as high as 200 could be handled with little equipment modification. Water use for the module was estimated at 12.4 US gpcd (47 lpcd) in a breakdown as shown in Figure 1. The potable water treatment equipment is capable of producing 15.2 lpcd but will normally operate at 7.6 lpcd.

The three treatment systems to be evaluated for the ESM have been previously identified as biological, physical-chemical and wet oxidation. The biological system would consist of grinding, aerobic biological treatment with powdered activated carbon, clarification, multimedia filtration and ozone oxidation/disinfection, followed by distillation of the effluent by thin film evaporation. This unit is based on the CABOS shipboard waste treatment designed by ORF for Environment Canada.[6]

The physical-chemical treatment unit selected consisted of screening, lime treatment, multimedia filtration, possibly ammonia stripping, activated carbon adsorption, ozone oxidation/disinfection and demineralization by reverse osmosis, with incineration of garbage, surplus sludges and the concentrate from the reverse osmosis step. The incineration is the same as that employed in the CANWEL treatment concept discussed in the second section of this paper.

The wet oxidation unit processes included maceration, wet oxidation, lime treatment, possibly ammonia stripping, multimedia filtration, reverse osmosis and ozone oxidation/disinfection. The macerated garbage and black water are wet oxidized for organic destruction. The effluent, after solids removal, is combined with the grey water and lime treated. The reverse osmosis and ozone units demineralize and disinfect.

A comparison of the three units as summarized by Figure 2 led to the recommendation of the wet oxidation concept for use in the ESM.

Wet Air Oxidation

Wet air oxidation is a process in which combustible material is oxidized by air or oxygen in the presence of liquid water. Elevated temperatures are required to achieve a useful reaction rate, and therefore to maintain a liquid phase, elevated pressures are required. The process is operated continuously with liquid and gas streams being introduced to the reactor simultaneously.

The waste is pumped via a high pressure pump through a heat exchanger into the reactor where it is mixed with compressed air. The oxygen in the air reacts with the organic matter in the slurry to produce mainly carbon dioxide and water. Due to the exothermic nature of the reaction, heat is produced which raises the temperature of the reactor contents. The effluent from the wet oxidation reactor is heat exchanged with the incoming waste to achieve autothermal operation.

The heart of the process is the WETOX reactor in which the oxidation takes place. It is a multi-compartmented, horizontal reactor with agitation and oxidant addition in each compartment. The violent agitation substantially increases the rate of oxidation. Within the WETOX reactor, the vapour and liquid phases are withdrawn separately. This feature increases the effective retention time of the liquid. The design of the WETOX process results in efficient oxidation at modest temperatures and pressures (e.g., 230°C, 600 psi), a fact which is extremely important to the economic viability of small continuous-flow applications, such as the ESM.

ESM Pilot Plant Evaluation

The second phase of the DCIEM ESM project was to conduct a pilot scale evaluation of the conceptual waste treatment system.[7] The pilot plant was designed and constructed at DCIEM. Long-term tests were performed on each of the unit processes to determine the optimum process conditions and to establish the expected performance of each. Test results to date indicate production of a water suitable for reuse.

The pilot plant tests[7] were conducted with simulated waste feeds. The feed material in the first waste simulation tests consisted of simulated grey water and simulated WETOX process effluent. This mixture was used for the evaluation of optimum conditions for the lime treatment and reverse osmosis stages. The second waste simulation study was carried out by wet air oxidation of sewage sludge which was considered to be consistent with a black water/solid waste mixture.

The sewage sludge was used to determine the process performance of the WETOX process over a wide range of conditions (e.g., temperature, pH, catalysts). Semi-batch tests were undertaken with typical process conditions such as 232°C and 600 psi. The pilot tests also produced a large volume of treated effluent for subsequent tests of the lime treatment, reverse osmosis and UV ozone oxidation/sterilization.

The results of the batch WETOX tests on the sewage sludge as shown in Figures 3, 4 and 5 illustrate the effect of varying the operating pressure, temperature, pH and catalysts. After 60 minutes in the batch WETOX reactor the overall COD reduction at 250°C was 74 per cent At 225°C, COD reduction was 69 per cent. Adjusting the pH to 3.0 resulted in an increased COD reduction to 83.5 per cent. The addition of cupric oxide (CuO) at 250 mg/l increased COD reduction to 91 per cent.

Using the optimum conditions from the batch WETOX tests, semi-batch tests were conducted at 232°C and 600 psi with raw sludge and raw sludge adjusted to pH 3 in conjunction with 250 mg/l CuO. These tests, as illustrated by Figure 6 show COD reductions of 75.0 per cent and 80.0 per cent for raw sludge and treated sludge respectively.

Pilot scale evaluations of sludge wet oxidation were conducted with a Model 4-10 continuous-flow WETOX pilot plant as shown in Figure 7. Tests were completed with a 52-minute residence time using temperatures of 219°C, 224°C and 227°C. The per cent reductions in total organic carbon (TOC) for these three tests are illustrated in Figure 8. Comparison of all the results from the batch, semi-batch and pilot plant tests are shown in Figure 9. A good correlation is obtained between all the sets of data, indicating that COD removals of approximately 75 per cent would be achieved under typical WETOX process conditions. Settling of the effluent from the process would increase the COD removal by a further 5 to 10 per cent.

Following the WETOX pilot tests, simulated grey water and WETOX. effluent were evaluated in the Lime Treatment Stage of the process. The pilot lime treatment system, as shown in Figure 10, consists of a lime slurry feed tank and pump, a rapid mixer, a flocculator and a clarifier. The loading to the pilot plant was 630 ml/min. The sludge was withdrawn from the clarifier at 40 ml/min and at a 1 per cent solids concentration.

Extended lime treatment tests were run at pH's of 9.5, 10.0, 10.5 and 11.0 to establish the optimum pH for clarification. The results of these tests are summarized in Table 1. At a pH of 10.0, reductions in suspended solids, total organic carbon and chemical oxygen demand were 95 per cent, 24 per cent and 20 per cent respectively. No substantial improvement was achieved at the higher pH values.

During a 2-week period of continuous operation of the lime treatment stage at a pH of 10.0, the COD of the waste was reduced from 1600 mg/l to 500 mg/l for an overall COD reduction of 70.0 per cent. Suspended solids reduction was in excess of 95 per cent.

Following lime treatment, the effluent was subjected to reverse osmosis (RO). Treatment was achieved with 3 RO units in series as shown by Figure 10. The first module was tubular and operated at 900 psi. The second unit was spiral wound and operated at 480 psi. The final unit was hollow fibre and operated at 190 psi.

The clarified effluent from the lime treatment stage was treated with hydrochloric acid (HCl) to adjust the pH to approximately pH 5.5. This was pumped to RO #1, the permeate from RO #1 being the feed to RO #2, and the permeate from RO #2 being the feed for RO #3. The concentrate streams from RO #1 and RO #2 were discharged. The concentrate from RO #3 was returned to the feed of RO #1.

The results from the RO tests are given in Table 2 for water recoveries of 75 per cent, 85 per cent and 90 per cent. Salt rejection efficiencies of 85 to 95 per cent were obtained. COD rejection efficiencies were in the order of 85 per cent, 75 per cent and 70 per cent at water recoveries of 75 per cent, 85 per cent and 90 per cent respectively. There was no evidence of membrane fouling even in RO #1 operating at 90 per cent water recovery.

Effluent from the reverse osmosis modules is pumped to an ultraviolet ozone reactor to ensure completion of oxidation and sterility of the recovered water. The addition of the ultraviolet to the ozone is reported by Prengle *et al*[1] to enhance oxidation 100 to 10,000 fold, thus driving oxidation reactions to completion in relatively short periods of time. The UV photochemically excites the molecules which are then oxidized by the ozone.

When combining UV and ozone, one must select the operating conditions carefully, because although increased temperature and increased ultraviolet light accelerate the ozone oxidation reactions, both conditions also accelerate the autodecomposition of the ozone. The ozonation of acetic acid is shown in Figure 11, with and without UV addition. Oxidation of the acetic acid with varying doses of UV is shown in Figure 12.

The pilot UV-Ozone reactor for the DCIEM tests as shown in Figure 13 was a 20.5 litre column with a water recirculation capacity of 227 lpm (50 gpm) to ensure maximum turbulence. Ozone flowrate could be set at 3.0, 6.0 and 10.0 grams per hour. Three UV lamps were utilized to provide 0.5 to 1.0 UV watts per litre.

Tests using the UV-Ozone column were first conducted with distilled water to determine the effect of temperature and ozone flowrate on the steady-state concentration of dissolved ozone. The results, as shown in Figure 14, indicate the steady-state dissolved ozone concentration was 3.0 mg/l, 1.5 mg/l and 1.0 mg/l for a 10 gram per hour input at operating temperatures of 25°C, 45°C and 50°C, respectively. The effect of the ultraviolet light on the steady-state ozone levels is illustrated in Figure 15. In general, 0.5 watts per litre UV reduces the steady-state ozone level by 50 per cent at all temperatures studied. At 1.0 watts per litre UV, the dissolved ozone concentration is reduced by 90 per cent.

Using the reverse osmosis permeate from the previous tests, the UV-Ozone contactor was evaluated at 45°C and 60°C for ozone flows of 6 grams/hour and 10.0 grams/hour at UV intensities of 0.5 and 1.0 watts/litre. The results of these tests are shown in Table 3. The results indicate that 45°C is the optimum temperature, and for an influent TOC of 33.5 mg/l at least 6 grams/hour are required. Depending on the ozone flowrate, UV intensities of 0.5 to 1.0 watts per litre may be sufficient. Maximum liquid residence time required is 3 hours if violent agitation is present.

ESM Final Design

Based on the Pilot Plant ESM tests a conceptual design of an air transportable unit was developed. Some modifications of the design were implemented after reviewing the experimental test results. The modified ESM, as described by DCIEM, has not been tested to date.[1] An outline of the ESM process and equipment to be used is as follows:

The glass and metallic refuse is collected separately from other solid waste and fed to a compactor with a volume reduction ratio of 15:1. The remaining soft solids are fed to a shredder-macerator which grinds the refuse to approximately 1 cm particle size, or to longer filaments. The effluent is mixed with the black water from the toilets and sent to a holding tank incorporating a submerged grinder pump which thoroughly macerates this stream to a slurry. Any glass or metallic particles which are inadvertently admitted to the shredder will not be drawn to the grinder, but will collect in the holding tank.

The grinder pump effluent is transferred to the wet oxidation reactor. This reactor would be approximately one-half the size of the one used in the pilot plant tests shown in Figure 7, and would incorporate three stirrers rather than the four shown. Additional equipment required for reactor operation would be a heat exchanger for transferring heat from the liquid wet oxidation effluent to the waste reactor feed stream, and a 2 HP air compressor.

After the heat exchanger, the reactor liquid phase effluent is mixed with the grey water and undergoes rapid mixing with lime followed by flocculation and clarification to remove suspended solids. The clarified effluent is passed through a cartridge filter to remove any residual solids, acidified to pH 5.5 for RO membrane protection, and sent to the two-stage RO unit to remove and concentrate soluble organics and inorganics. The first RO unit employs a tubular membrane and operates at a 90% water recovery rate. The concentrate from the first RO unit is mixed with the steam phase from the reactor and fed to an evaporator for volume reduction and concentration of this waste stream. The permeate from the first RO unit flows to the second RO unit which employs a hollow fibre membrane and operates at a water recovery rate of 75%. The concentrate from this second RO unit is recycled to the feed of the first RO unit. The permeate from the second RO unit flows to a holding tank before being processed in a batch operation by the UV/ozone contactor which destroys any residual organics. The resultant water is of potable quality, and is recycled within the ESM.

The per capita water use of 47 litres/day has been made possible through the use of several water conserving peripheral facilities in the ESM. Water use as a carrier in the collecting sewer has been reduced by incorporating a vacuum sewer system in which slugs of wastewater are transported by air pressure rather than gravity. A special toilet has been developed for the vacuum sewer system which uses approximately one-tenth as much water as conventional toilets. Water consumption in the showers has been reduced by incorporating saunas into the ESM unit and the installation of flow limiting shower heads.

The layout of the complete ESM is illustrated in Figure 16 with a breakdown of the treatment system and peripheral facilities shown in Figure 17 and Figure 18 respectively. Costs of the ESM unit have been compared to waste treatment for the community of Igloolik in the Canadian Northwest Territories, which uses a primitive waste disposal system. The individual costs of the two waste disposal processes are shown in Table 4 and Table 5. Using this information and assuming 150 people in the community, the Igloolik services would cost $330 per day. The corresponding ESM facility has a daily cost of $318 per day, based on $257,000 capital costs amortized over 15 years at 9 per cent, electrical expenses of $87 per day and operation and maintenance costs of $147 per day. The $147 operation and maintenance cost is based on work conducted for the Alaska Village Demonstration Project.[9]

A summary of the cost comparison is provided in Figure 19. It must be stressed in this comparison that for comparatively equal costs two distinctly different types of service are being considered. The existing service systems provide water and waste removal to and from homes with no treatment of the wastes. The ESM installation would require the use of a centralized facility in order to completely treat all the waste streams. In addition, the ESM provides a more environmentally acceptable disposal residue.

DEVELOPMENT OF THE CANADIAN WATER ENERGY LOOP (CANWEL)

Design Integration

The CANWEL system is a more conventional unit than the ESM process previously discussed. The liquid waste portion of the CANWEL system includes biological nitrification-denitrification, phosphorus precipitation, granular activated carbon filtration, ozone disinfection and reverse osmosis.[10] The solid waste treatment unit consists of starved air incineration for destruction of solid waste (household garbage), surplus biological and chemical sludge, and reverse osmosis brine.[11] The integrated CANWEL system, as illustrated in Figure 20, has been designed to treat all liquid and solid wastes for a community, to the point where liquid and gaseous discharges from the system will exert a minimum load to the environment. In achieving this high quality effluent the system produces a water quality suitable for reuse as utility water.

An installation of the liquid waste treatment portion of a CANWEL unit can use one or two subsections, depending on the degree of water quality required. For undiluted surface discharge, removal of carbon, nitrogen and phosphorus is essential, together with effective disinfection of the effluent for health protection. For reuse, however, the additional steps of activated carbon filtration and reverse osmosis are required. The effluent quality achievable to date is compared to the Canadian objectives in Table 6.

The CANWEL approach is presently being considered for high rise apartment buildings where treatment and recycle can be utilized in conjunction with the solid waste treatment unit. CANWEL also has applications in towns and subdivisions where facilities do not exist for dilution of a poorer quality effluent. At present the apartment building concept is being studied in Toronto, Ontario, Canada, where a totally integrated 500 people unit has been constructed in the subbasement of an apartment building and is now awaiting commissioning.

Nitrification-denitrification is carried out in a patented two-stage system without the use of an added carbon source.[12] As shown in Figure 21, raw sewage first enters an anoxic biological reactor and is then airlifted to an aerobic biological reactor where the ammoniacal nitrogen waste is oxidized to nitrate-nitrogen. This mixed liquor, as shown in Figure 22, is then recycled back to the first reactor where it meets the incoming carbon in the raw sewage under anoxic conditions suitable for denitrification. Efficient operation of the nitrification -denitrification unit requires precise control of the internal recycles and the reactor dissolved oxygen levels.

The mixed liquor, having low concentration of carbon and nitrogen, then passes to a sludge separator. Owing to the nature of the biological treatment a poor settling sludge is developed; however, the design of the settler allows for easy solids separation. As shown in Figure 23, the mixed liquor enters the separator tangentially at three points which results in a slow rotation of the settling suspension. The settling rate is further enhanced by the sloping surface of the inner settling cone. The settled sludge is returned at a rate of 300 per cent to the second biological reactor.

Effluent from the sludge separator overflows to the chemical reactor-clarifier for removal of phosphorus and residual suspended solids. As shown in Figure 24, alum is added to a central mixing well where it is mixed with biological effluent and recycled chemical sludge. The internal recycle rate must be set to maintain the design settling rate of the clarifier and optimize the use of the alum.

After chemical treatment the effluent is disinfected with ozone prior to discharge. The effluent may also pass to carbon filtration and/or reverse osmosis if a higher quality recycle effluent is required.

To date the ORF has concentrated on producing a high quality effluent for undiluted surface discharge and thus less emphasis has been placed on the demonstration of the reverse osmosis module.

Ozone disinfection was selected for use in the CANWEL system primarily for efficacy and ease of on-site generation. The disinfection is carried out in a counter-current reactor at a dose of 4.4 mg of ozone per litre of effluent. The residual ozone level after 2 hours in a final storage tank is in the range of 0.1 mg/l. With disinfection preceded by carbon filtration, bacterial growth is normally non-detectable. Without carbon filtration, however, the standard plate count is around 5 colonies/ml.

CANWEL Pilot Plant Evaluation

Two intensive pilot scale tests were completed in 1976 and 1977 with a 10,800 Imp. gpd (49,096 lpd) test unit. During 1976 the pilot system was operated for three months without any modifications, and grab samples were taken daily from each point of liquid exchange between the reactors. Composite influent and effluent samples were also collected during this period. In 1977 the nitrification-denitrification system was evaluated under three different hydraulic loadings (22.7 lpm, 34.1 lpm and 45.5 lpm) in combination with a variation in the biological reactor recycle rates. At the completion of these tests the unit was operated under variable flow conditions for a four-week period with two weeks of hourly sampling, 24 hours per day.

The data collected for the 3-month 1976 run are given in Table 7 as mean grab and composite values for 36 sampling days. The high degree of nitrification-denitrification is illustrated by the low levels of 0.1 mg/l ammonia-nitrogen (NH_3-N) and 5.1 mg/l nitrate-nitrogen (NO_3-N). The effluent phosphorus is essentially all soluble orthophosphate (SPO_4-P) and is less than 0.1 mg/l as phosphorus. Reduction of organics is very high as illustrated by the drop in Total Organic Carbon (TOC) from 113 mg/l to 5.8 mg/l.

The fraction of each contaminant remaining in the grab samples after each unit process is given in Table 8. This information shows an 89 per cent total nitrogen removal, 99 per cent total phosphorus removal and 95 per cent total organic carbon removal. The composite samples show removal efficiencies of 83 per cent for nitrogen and 98 per cent for 5-day biochemical oxygen demand (BOD_5).

High values for the biological effluent suspended solids are due to the self-regulating aspect of the sludge separator. No solids are intentionally wasted from this unit, instead they flow over into the reactor-clarifier when necessary. All sludge from the reactor-clarifier is then airlifted to a thickener. The biological effluent suspended solids ranged from 5 mg/l to 4,500 mg/l during this study.

Microbiological data for this period are shown in Table 9. The final effluent was well within the guidelines of the Canadian Drinking Water Standards[13], and effluent from the first stage ozonation was well within permissible values for Ontario Public Surface Waters.[14]

The design parameters for the CANWEL system during the 3-month test are shown in Table 10. Comparative values are provided for an extended aeration plant and a conventional activated sludge system.

Additional design parameters which also must be considered for the CANWEL process are the removal rates for the various forms of nitrogen. The removal rates for the three month study are shown in Table 11. An average nitrogen removal rate of 0.031 kilograms of nitrogen per kilogram of mixed liquor volatile solids per day (kg/kg MLVSS/day) is high compared to the rate of 0.016 kg/kg MLVSS/day for an alternating air nitrification-denitrification system.[15]

The data collected for the steady-state operating conditions during the 1977 loading variation studies are given in Table 12. For this study only the biological units and chemical reactor-clarifier were operated. A poorer quality effluent than that found in the 1976 steady-state study is to be expected owing to the number of operational changes made to the system over a short time interval. The results indicate that there was not a significant change in the effluent quality due to increased nitrogen or organic carbon loading. A slight deterioration in phosphorus and suspended solids levels occurred due to hydraulic overloading of the reactor-clarifier.

The nitrogen and carbon removal rates for this study, as illustrated in Table 13, suggest that the plant was not organically overloaded at 10 gpm (45.5 lpm) as had been expected. Per cent removal of nitrogen dropped less than 5 per cent with a doubling of nitrogen loading. There was no reduction in per cent removal of organic carbon.

In order to study the effect of daily flow variation, a flow recorder was installed in the main sewer feeding the pilot plant, and based on an analysis of these data, a pilot flow scheme was set up as shown in Table 14. This allowed for 9,990 gpd (45,415 lpd) with a high peak to average ratio of 1.44 and a low peak to average ratio of 0.22. The internal recycle rates were set at the values found optimum for the 10.0 gpm (45.5 lpm) study.

The results of this program, provided in Table 15, illustrate the ability of the biological unit to operate under daily flow fluctuation while still maintaining a relatively good effluent quality. The nitrogen levels in the effluent, however, are higher than those found in the 1976 steady-state study and would not meet surface discharge requirements.

The reason for the higher nitrogen levels is the fluctuation in the dissolved oxygen levels in the biological reactors as shown in Figure 25. This has been brought about by the uneven organic loading. Raw Sewage BOD_5, TN and TOC over a typical 24-hour period are shown in Figure 26. The peaks in this graph are further amplified by the peaks in the hydraulic loading. This results in a large variation in the nitrogen levels within the biological reactors, as illustrated by Figure 27.

In order to control extreme variations in dissolved oxygen levels which allow complete nitrification at night with little denitrification, the CANWEL demonstration plants have been equipped with automatic dissolved oxygen controls. Dissolved oxygen levels will be maintained at less than 0.5 mg/l in the top of the anoxic reactor and less than 2.0 mg/l in the top of the aerobic reactor. Addition of this equipment should result in a considerable improvement in the effluent quality.

SUMMARY

The Ontario Research Foundation (ORF) has been actively involved in the development of two different wastewater treatment systems designed to renovate wastewater to a quality suitable for reuse.

One system is designed for military use as an Environmental Service Module (ESM) and the other for domestic application as part of an integrated waste management system called CANWEL (Canadian Water Energy Loop).

The military ESM is an air-transportable, combined wastewater purification, solid waste disposal system complete with facilities for 100-200 men for operation in the north. The process which employs wet air oxidation, reverse osmosis and ozone-UV oxidation is designed to treat all solid wastes (garbage) and human wastes (sewage) and produce a residue which is sterile, compact and may be disposed of relatively easily and safely. Extensive water recycling (80%) is employed for purposes other than drinking and culinary use.

The CANWEL system employs biological nitrification-denitrification, phosphorus removal, ozone disinfection and reverse osmosis steps for wastewater renovation. A starved air incineration method for disposal of garbage, surplus sludge and brine disposal is employed with heat recovery. The system has been designed to treat all liquid and solid wastes from a community of 500-1,000 people to a degree where liquid and gaseous discharges from the system will exert a minimum load to the environment. The degree of purification achieved will permit reuse of water for utility purposes.

ACKNOWLEDGMENTS

Most of the work on the ESM reported herein is taken from reports by A.P. Cadotte, Department of Environmental Chemistry, Ontario Research Foundation, Mississauga, Ontario, Canada, L5K 1B3.

Work conducted at ORF on the ESM was sponsored by the Defence and Civil Institute of Environmental Medicine, Department of National Defence - Canada, 1133 Sheppard Avenue West, P.O. Box 2000, Downsview (Toronto), Ontario, Canada, M3M 3B9.

Research & Development on the CANWEL system was sponsored by the Central Mortgage and Housing Corporation, Ottawa, Ontario, Canada.

REFERENCES

1. DAY, D. Research and Development Towards a Facility for Water Purification and Waste Treatment in the North. Defence and Civil Institute of Environmental Medicine, Toronto, Ontario, Canada (1978).

2. DAY, D. Development of an Environmental Service Module for Water Purification and Waste Disposal in the North. Defence and Civil Institute of Environmental Medicine, Toronto, Ontario, Canada (1977).

3. BROWN, C. K.; CADOTTE, A. P.; SETO, P. & SMITH, D. K. Conceptual Design of an Environmental Service Module. Ontario Research Foundation Report 75-01. Mississauga, Ontario, Canada (1975).

4. ARD, R. W. JR. & SCARANO, T. S. Marine Wastewater Treatment Systems. Env. Sci. Tech., 8:219 (1974).

5. VAN HEES, W. & CONSTANT, A. E. Shipboard Sanitary Waste Treatment/Management Systems. Paper presented at the Intersociety Conference on Environmental Systems, San Diego, Ca. (1973).

6. LOMAS, H. & TOWNSHEND, A. R. CABOS - A New Wastewater Treatment System for Vessels. Report EPS4-WP-76-2, Environment Canada, Ottawa, Ontario, Canada (1976).

7. CADOTTE, A. P. ESM Pilot Plant Wastewater Treatment System Evaluation - Final Report. Ontario Research Foundation, Mississauga, Ontario, Canada (1978).

8. PRENGLE, et al. Ozone-U.V. Process Effective Wastewater Treatment. Hydrocarbon Processing, 54:10:82 (1975).

9. PUCHTLER, B.; REID, B. & CHRISTIANSON, C. Water-Related Utilities for Small Communities in Rural Alaska. Report #EPA-600/3-76-104 U.S. EPA, Corvallis, Oregon (Sept. 1976).

10. LAUGHTON, R. V. CANWEL 10,000 GPD Wastewater Treatment Plant - Process Studies and Testing. Report 76-11, Ontario Research Foundation, Mississauga, Ontario, Canada (1976).

11. CANWEL - A Canadian Waste Management System. Central Mortgage and Housing Corporation, Ottawa, Ontario, Canada (1977).

12. LAUGHTON, R. V. U.S. Patent Application 916,868 (now granted). Ontario Research Foundation, Mississauga, Ontario, Canada.

13. Canadian Drinking Water Standards and Objectives. Department of National Health and Welfare, Information Canada, Ottawa, Ontario, Canada (1970).

14. Guidelines and Criteria for Water Quality Management in Ontario. Ontario Ministry of the Environment, Toronto, Ontario, Canada (1976).

15. BISHOP, D. F.; HEICHMAN, J. A. & STAMBERG, J. B. Single Stage Nitrification-Denitrification. Jour. WPCF, 48:3:520 (1976).

Note: WETOX and CANWEL are registered trademarks.

TABLE 1

Lime Treatment Tests on ESM WETOX Waste Simulation

Sample	Suspended Solids mg/l	Total Organic Carbon mg/l	Chemical Oxygen Demand mg/l
Feed	925	1,400	3,100
Clarified Effluent at pH = 9.5	70	1,000	2,300
Clarified Effluent at pH = 10.0	30	1,000	2,300
Clarified Effluent at pH = 10.5	28	1,000	2,300
Clarified Effluent at pH = 11.0	26	1,000	2,300

TABLE 2

Reverse Osmosis Tests on ESM Waste Simulation

Test Number	Per Cent Water Recovery on Each RO Unit	Sample	Conductivity (μ mhos/cm^2)	TOC mg/l	COD mg/l
		Feed	2,000	150	500
1	75%	RO #1 Permeate	300	50	150
		RO #1 Concentrate	5,000	400	1,000
		RO #2 Permeate	100	30	100
		RO #2 Concentrate	500	50	150
		RO #3 Permeate	25	25	65
		RO #3 Concentrate	300	50	150
2	85%	RO #1 Permeate	700	45	110
		RO #1 Concentrate	9,300	650	1,350
		RO #2 Permeate	250	25	80
		RO #2 Concentrate	2,500	135	230
		RO #3 Permeate	30	15	55
		RO #3 Concentrate	1,500	90	230
3	90%	RO #1 Permeate	750	60	160
		RO #1 Concentrate	11,800	825	1,900
		RO #2 Permeate	400	40	120
		RO #2 Concentrate	4,000	225	525
		RO #3 Permeate	85	25	80
		RO #3 Concentrate	3,300	125	450

TABLE 3

U.V.-Ozonation of ESM RO Permeate

Test Conditions			TOC Concentration mg/l							
Temperature (°C)	Ozone Flow (g. hr^{-1})	U.V. Intensity (watts. l^{-1})	1 hr	2 hr	3 hr	4 hr	5 hr	6 hr	7 hr	8 hr
45	-	0.5	-	-	-	-	-	-	-	33.5
45	-	1.0	-	-	-	-	-	-	-	33.5
45	10.0	-	-	-	-	-	-	-	11.5	11.0
45	6.0	0.5	-	-	-	-	-	11.5	-	-
45	6.0	1.0	-	12.0	4.0	2.5	-	-	-	-
45	10.0	0.5	4.5	-	4.0	3.5	-	-	-	-
45	10.0	1.0	5.0	-	2.5	-	-	-	-	-
60	6.0	0.5	19.0	10.0	8.0	4.0	-	-	-	-
60	6.0	1.0	11.0	6.5	-	-	5.0	-	-	-
60	10.0	0.5	6.0	5.5	-	-	3.5	2.0	-	-
60	10.0	1.0	-	-	-	-	-	13.0	-	-

NOTE: Influent TOC = 33.5 mg/l

TABLE 4

Waste Treatment Costs, Igloolik, Canada

Electricity:	25¢/kwh
Fresh Water Delivery and Grey Water Removal:	2.2¢/l of fresh water delivered to holding tanks in homes. While of fair quality, because of frequent handling this water is always boiled before being used for drinking purposes. Grey water consists of dishwater and bathing and laundry waste water, and the cost of its removal from holding tanks in homes is included in this charge.
Human Waste Removal:	$1.25/honeybag removal by truck. The honeybag is a simple plastic bag receptacle used in toilets in homes, and lasts approximately six man-days.
Solid Waste (Garbage) Removal:	$1.25/45 gallon drum

TABLE 5

Estimated ESM Capital Costs

Shredder	$ 5,000
Pump Grinder	2,000
Wet-oxidation Reactor	120,000
Rapid Mixer, Flocculator, Clarifier	8,000
Evaporator	5,000
Reverse Osmosis Unit (tubular)	25,000
Reverse Osmosis Unit (hollow fibre)	3,000
Air Dryer, Ozone Generator, UV-Ozone contactor	25,000
3 Hercules Transportable Camp Units	30,000
Peripheral Facilities	5,000
Control Systems, Misc. Hardware, etc.	10,000
Fresh Water Treatment (e.g., Evaporator, UV Sterilizer)	9,000
Engineering Design	10,000
Total	$257,000

TABLE 6

Effluent From CANWEL Wastewater Treatment System (Reverse Osmosis Excluded)

Parameter		Canadian Drinking Water Standards	Ontario Public Surface Waters	CANWEL Waste Water Renovation Process	
		Acceptable Limits		Objective	Achieved to Date
Temperature	°C	15	29.5	30	25
Dissolved oxygen	mg/l			> 5.0	7.0
BOD_5	mg/l			3	3
Total solids	mg/l			500	500
Suspended solids	mg/l			5	3
Dissolved solids	mg/l	1000	500		
Total Phosphorus	mg/l	0.07		0.10	0.10
Soluble Phosphorus	mg/l			0.05	0.05
NH_3 as N	mg/l	0.50	0.50	0.10	0.10
$NO_3 + NO_2$ as N	mg/l	10.0	10.0	3.0	4.0
Chloride	mg/l	250	250	150	70
pH		6.5 - 8.3	6.0 - 8.5	6.5 - 8.5	7.0
Turbidity	NTU	5		5	< 1
Colour	TCU	15	75	15	15
Odour	TON	4	removable	absent	absent
Total Coliform (MF)	org/100 ml	*	5000	1000	zero**
Fecal Coliform (MF)	org/100 ml		500	100	zero**
Enterococci (MF)	org/100 ml			20	zero**
Standard Plate	Count/l				< 10

* 95% of 30 day samples negative; positive counts not to exceed 4/200 ml or 10/500 ml

** 1 litre samples examined after neutralization of free ozone residual with thiosulphate

TABLE 7

Mean Water Quality During 1976 3-Month Test (36 Sampling Days)

Stream	NH_3-N mg/l	NO_3-N mg/l	NO_2-N mg/l	TKN mg/l	TN mg/l	TP mg/l	SPO_4-P mg/l	TOC mg/l	SOC mg/l	SS mg/l	VSS mg/l	BOD_5 mg/l
Raw Sewage Grab	25.3	0.26	0.08	46.9	47.3	7.3	4.8	113	56	165	138	-
Biological Effluent Grab	2.1	4.92	0.16	8.3	13.4	5.0	2.3	-	10	470	322	-
Clarifier Effluent Grab	0.4	4.19	0.04	1.0	5.2	0.2	0.05	8.7	-	5.5	2.0	-
Final Effluent Grab	0.1	5.06	ND	0.3	5.4	0.1	0.09	5.8	-	-	-	-
Raw Sewage Composite	23.2	1.72	0.34	28.9	31.0	4.9	3.3	81	31	120	94	204
Final Effluent Composite	0.18	4.75	<0.01	0.52	5.3	0.1	0.08	4.4	-	-	-	3.0

TABLE 8

Removal Capabilities of CANWEL Sub-Units

	Per Cent Remaining			
	Total Nitrogen	Total Phosphorus	Soluble Ortho Phosphate	Total Organic Carbon
Raw Sewage	100	100	100	100
Biological Effluent	18	68	48	-
Clarifier Effluent	11	3	1	8
Final Effluent	11	1	2	5

TABLE 9

1976 3-Month Run Microbiological Data
(Micro-organisms/100 ml)

Total Count: 0.22 μ Filter
Total Coliform: 0.45 μ Filter

(1 litre samples examined after neutralization of free ozone residual with thiosulphate)

	Sample	Chemical Clarifier Effluent	1st Stage Ozone Effluent	Carbon Filter Effluent	Final Effluent
Total Coliforms	1				0.6
	2			1,300	ND
	3			120	ND
	4		100	1,600	ND
	5		ND	1,700	2.0*
	6		10	780	ND
	7		40	102	ND
	8		30	30	ND
	9		50	20	ND
	10	22,000	100	550	ND
	11	24,000	30	150	1.5
	12	29,000	50	80	ND
Standard Plate Count	1				> 50
	2			8,000	ND
	3			700	0.1
	4		500	2,600	ND
	5		800	2,800	> 50*
	6		80	990	ND
	7		90	150	ND
	8		160	260	ND
	9		250	200	1.4
	10	38,000	80	1,000	0.6
	11	77,000	200	670	> 50
	12	90,000	830	560	0.9

* Signifies ozone generator failure ND-None detected

TABLE 10

Comparison of CANWEL Design Parameters
to Other Activated Sludge Processes

Parameter	Conventional	Extended Aeration	CANWEL
Mean Cell Residence Time (θc, days^{-1})	5 - 15	20 - 30	10 - 20
Organic Loading (kg BOD_5/kg MLVSS/day)	0.2 - 0.4	0.05 - 0.15	0.1 - 0.34
Mixed Liquor Suspended Solids (MLSS, mg/l)	1500 - 3000	3000 - 6000	3500 - 5000
Biological Retention Time (V/Q, hrs)	4 - 8	18 - 36	7.2
Sludge Recycle (Q^R/Q)	0.25 - 0.5	0.75 - 1.5	3.0
Oxygen Consumption (kg O_2/kg BOD removed)	0.8 - 1.1	1.6 - 3.3	0.9 - 3.3

TABLE 11

Nutrient Removal Rates for the CANWEL System

Parameter	Removal Rate (kg/kg MLVSS/day)
NH_3-N	0.020
Total N	0.031
SOC	0.040

TABLE 12

Water Quality Under Different Loading Conditions, 1977

Flow	Stream	NH_3-N (mg/l)	NO_3-N (mg/l)	NO_2-N (mg/l)	TKN (mg/l)	TN (mg/l)	TOC (mg/l)	SOC (mg/l)	TP (mg/l)	SPO_4-P (mg/l)	BOD_5 (mg/l)	SS (mg/l)
5.0 gpm (22.7 lpm)	Raw Sewage	25.7	1.13	0.11	76.0	77.3	133	69	9.4	5.5	190	121
	Biological Effluent	0.1	11.1	0.08	1.1	12.4	11	10	5.3	5.1	7.3	6.2
	Clarifier Effluent	<0.1	11.4	0.02	0.7	12.1	8	7	0.5	0.09	-	13
7.5 gpm (34.1 lpm)	Raw Sewage	18.2	1.04	0.11	56.7	58.9	116	69	9.6	4.2	-	138
	Biological Effluent	0.2	10.4	0.49	1.1	11.8	12	9	3.3	3.2	4.3	8.4
	Clarifier Effluent	<0.1	8.37	0.03	0.5	9.0	8	7	0.3	0.05	-	8.1
10.0 gpm (45.5 lpm)	Raw Sewage	34.1	0.32	0.11	64.3	64.8	153	82	9.9	5.0	209	147
	Biological Effluent	<0.1	11.9	0.15	1.1	13.2	11	10	4.2	4.2	9.4	10
	Clarifier Effluent	<0.1	11.3	<0.01	0.6	11.9	10	9	0.7	0.3	-	10

TABLE 13

Removal of Nitrogen and Carbon Across the Biological Unit Under Different Loading Conditions, 1977

Flow (gpm)	Total Nitrogen Removal Rate (kg/kg MLVSS/day)	Total Organic Carbon Removal Rate (kg/kg MLVSS/day)	Per Cent Removals	
			TN	TOC
5.0	0.041	0.076	84.0	91.5
7.5	0.048	0.121	79.6	90.2
10.0	0.096	0.264	79.5	92.5

TABLE 14

Feeding Cycle for Daily Flow Variation Study

Time	Flow (gpm)	Total Flow (Imp. Gallons)	Per Cent of Daily Flow
12 pm - 6 am	1.5	540	5.4
6 am - 1 pm	10.0	4,200	42.1
1 pm - 5 pm	7.0	1,680	16.8
5 pm - 12 pm	8.5	3,570	35.7

TABLE 15

Mean Water Quality During the 1977 Daily Flow Variation Study

Sample	NH_3-N (mg/l)	NO_3-N (mg/l)	NO_2-N (mg/l)	TKN (mg/l)	TN (mg/l)	TOC (mg/l)	SOC (mg/l)	BOD_5 (mg/l)	SS (mg/l)
Grab Sample Raw Sewage	35.5	0.36	0.07	63.1	63.5	132	78	170	118
Grab Sample Biological Effluent	0.3	7.2	0.15	1.2	8.6	14	11	4.1	3
Composite Sample Raw Sewage	17.6	0.51	0.02	38.0	38.5	122	67	155	110
Composite Sample Biological Effluent	0.8	7.1	0.15	2.2	9.4	13	12	10	22

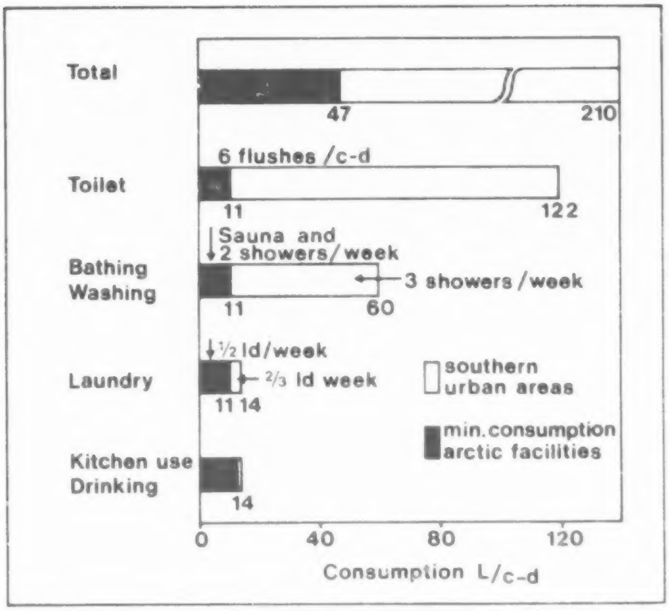


FIGURE I. WATER CONSUMPTION RATES FOR URBAN AREAS AND THE ESM.

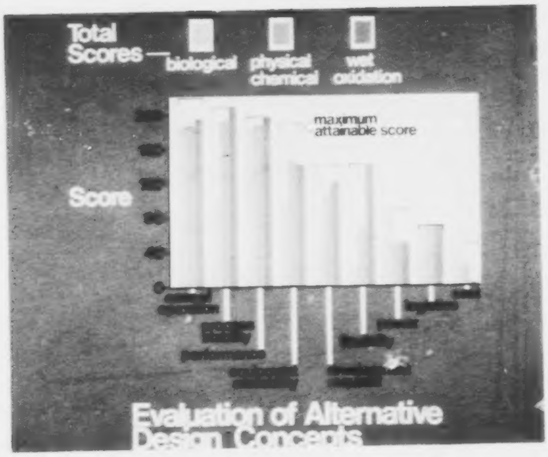


FIGURE 2. EVALUATION OF ALTERNATIVE DESIGN CONCEPTS FOR AN ESM.

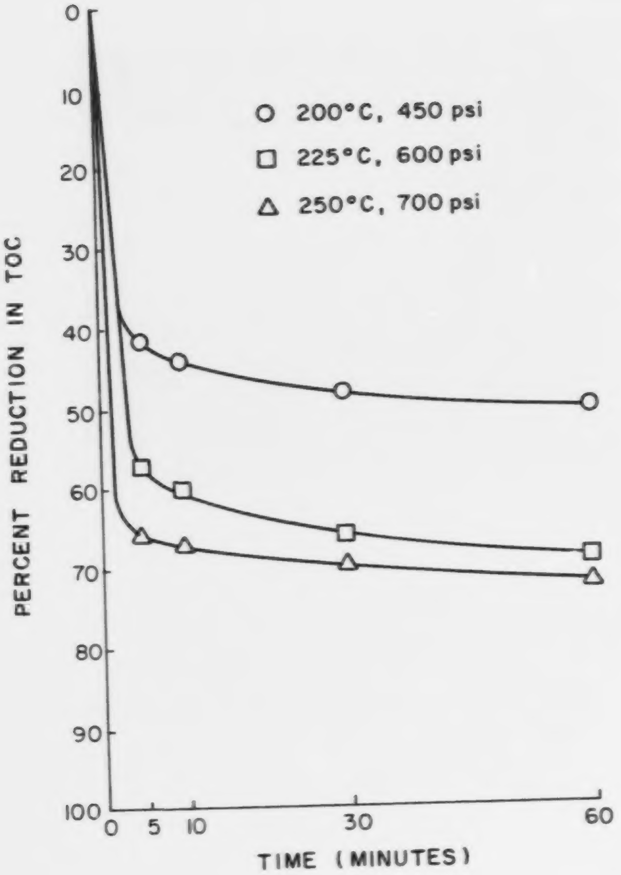


FIGURE 3. EFFECT OF TEMPERATURE ON ESM WETOX BATCH TESTS (CADOTTE 1978)

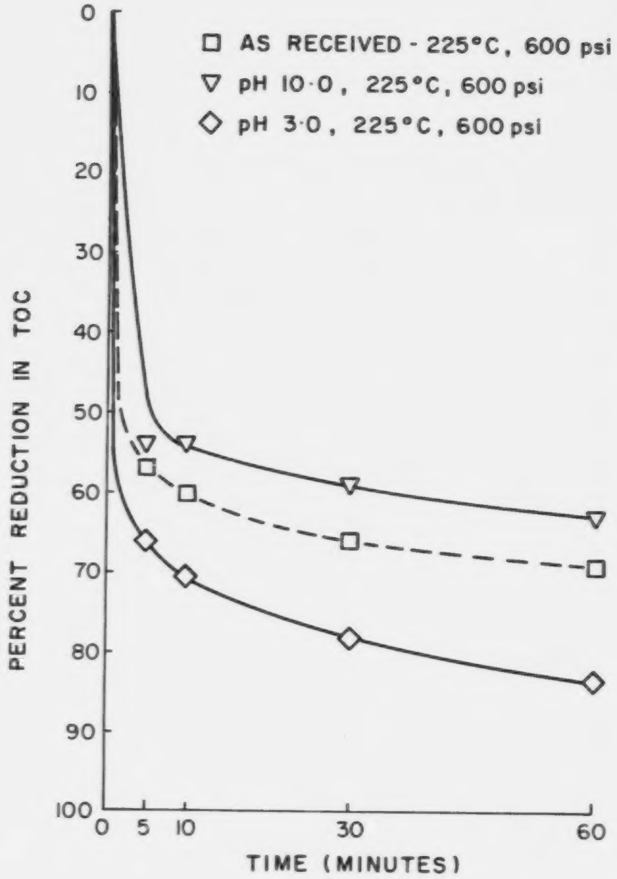


FIGURE 4. EFFECT OF pH ON ESM WETOX BATCH TESTS. (CADOTTE 1978)

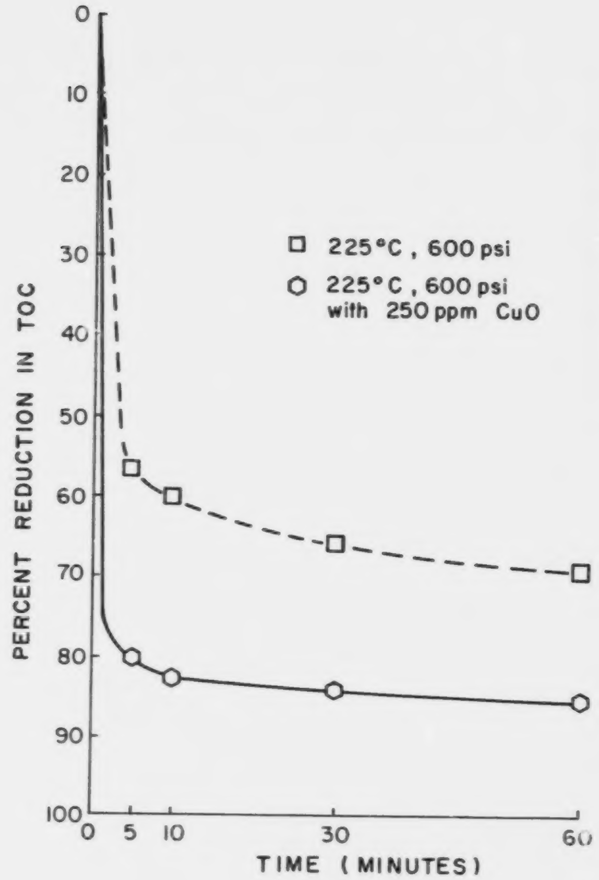


FIGURE 5. EFFECT OF CATALYSTS AT 225°C ON ESM WETOX BATCH TESTS. (CADOTTE 1978)

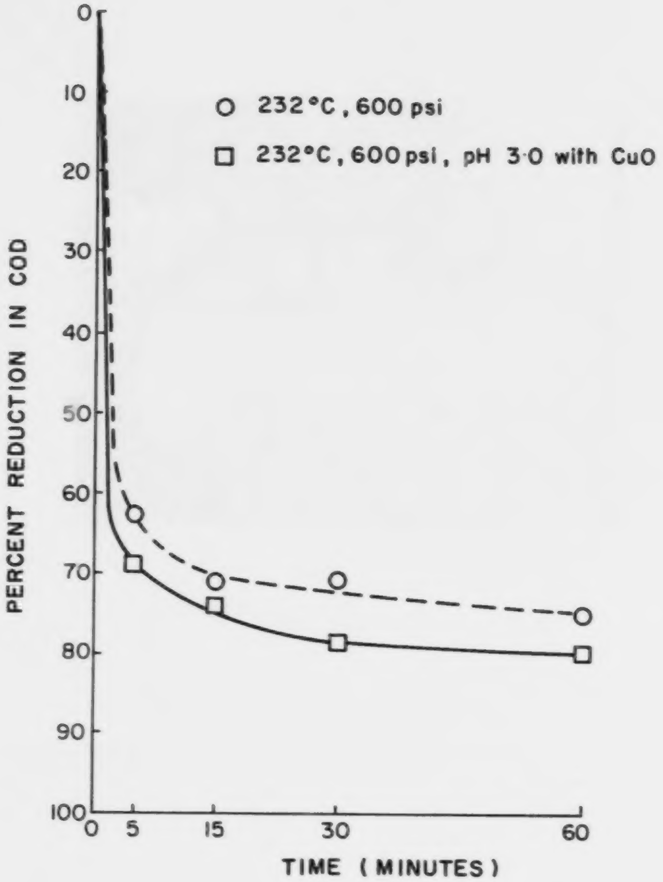


FIGURE 6. EFFECT OF pH & CATALYSTS ON ESM WETOX SEMI-BATCH TESTS. (CADOTTE 1978)

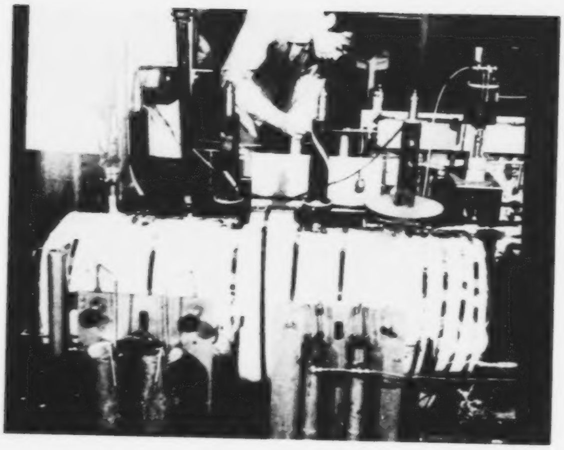


FIGURE 7. MODEL 4-10 WETOX PILOT PLANT USED FOR ESM EVALUATION.

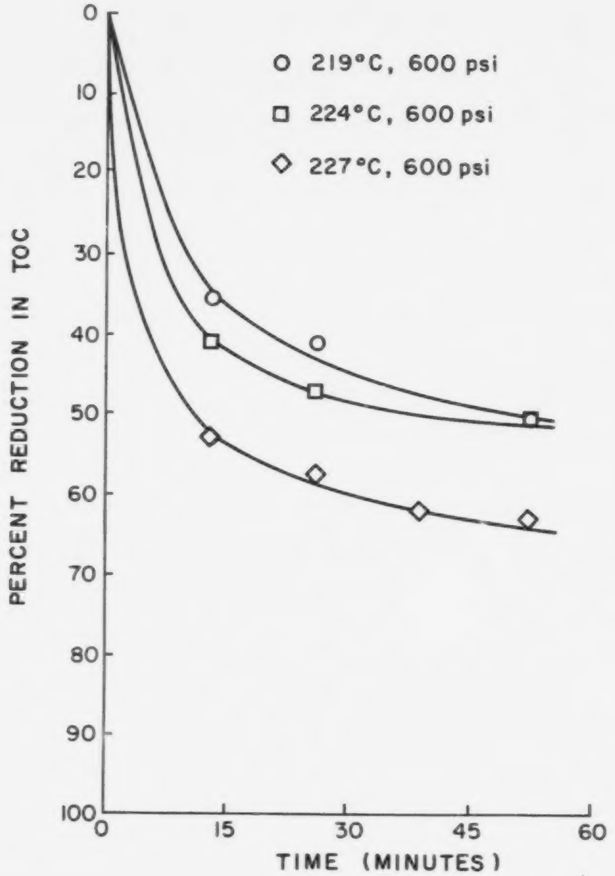


FIGURE 8. EFFECT OF TEMPERATURE ON ESM WETOX PILOT PLANT TESTS. (CADOTTE 1978)

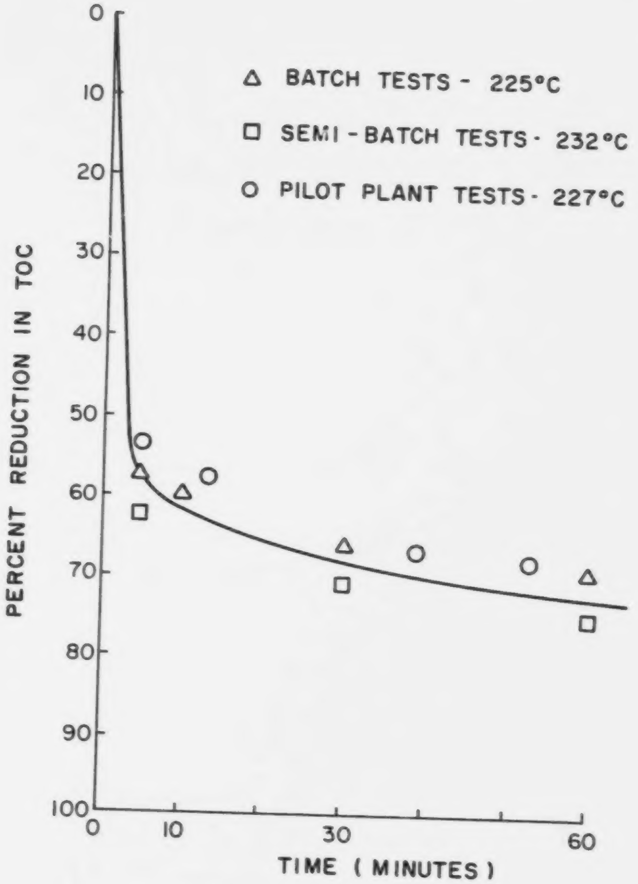


FIGURE 9. COMPARISON OF ESM WETOX BATCH, SEMI-BATCH & PILOT PLANT RESULTS AT 225° - 232°C. (CADOTTE 1978)

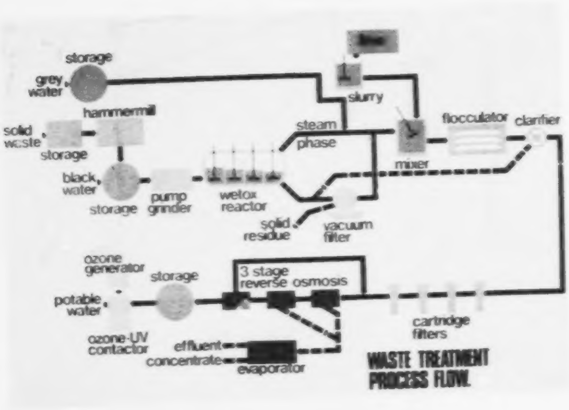


FIGURE 10. ESM TOTAL TREATMENT SCHEME.

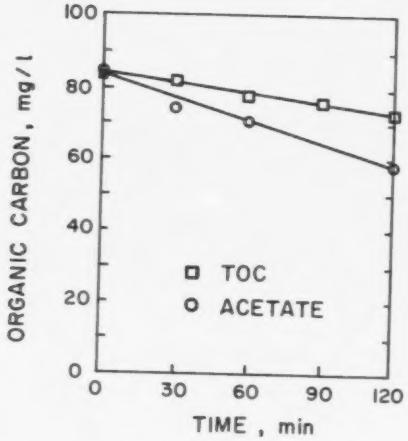


FIGURE 11a OZONATION OF ACETIC ACID

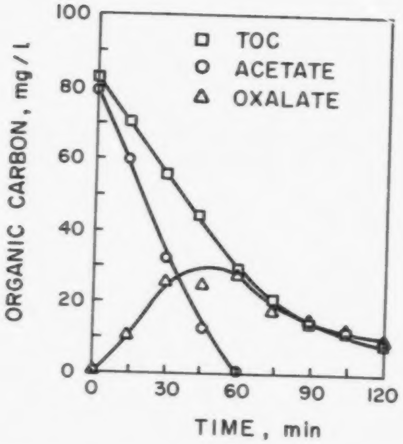


FIGURE 11b U.V.-OZONATION OF ACETIC ACID

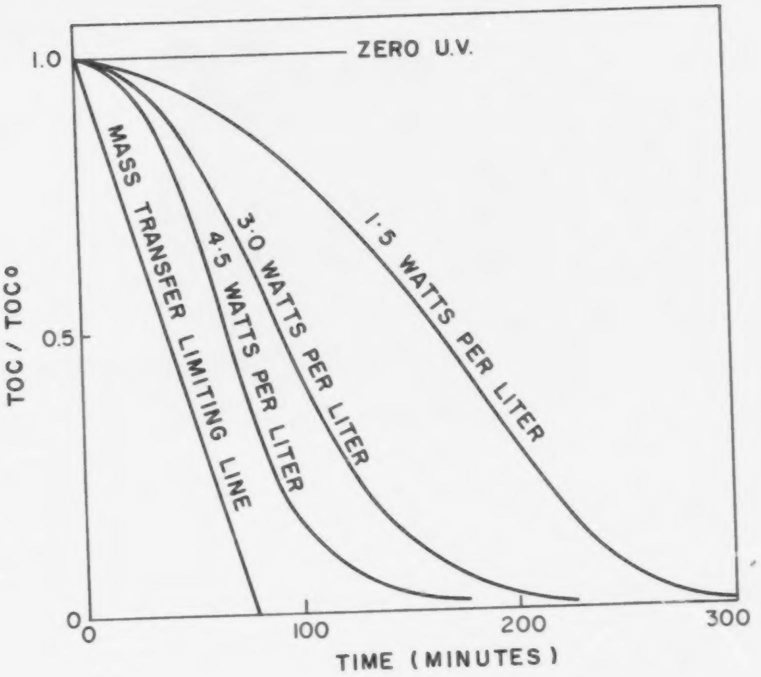


FIGURE 12. U.V. OZONATION OF ACETIC ACID.
(PRENGLE 1975)

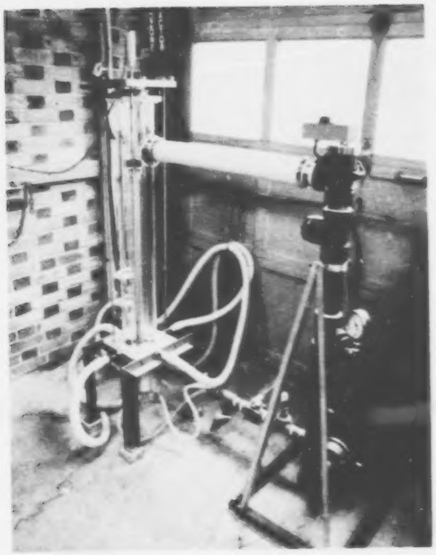


FIGURE 13. PILOT ESM U.V-OZONE COLUMN.

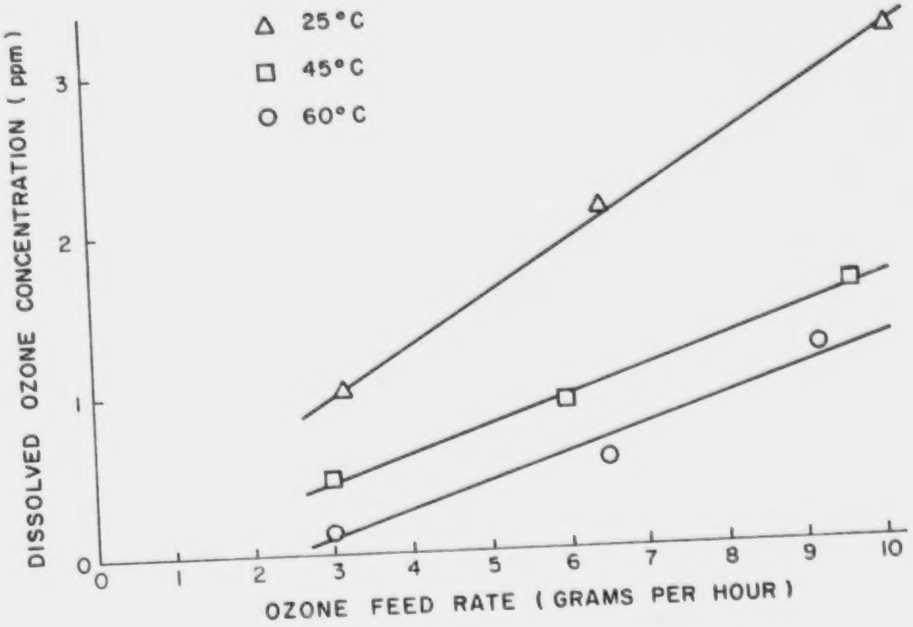


FIGURE 14. EFFECT OF TEMPERATURE AND OZONE FLOW RATE ON THE STEADY-STATE OZONE CONCENTRATION (CADOTTE 1978)

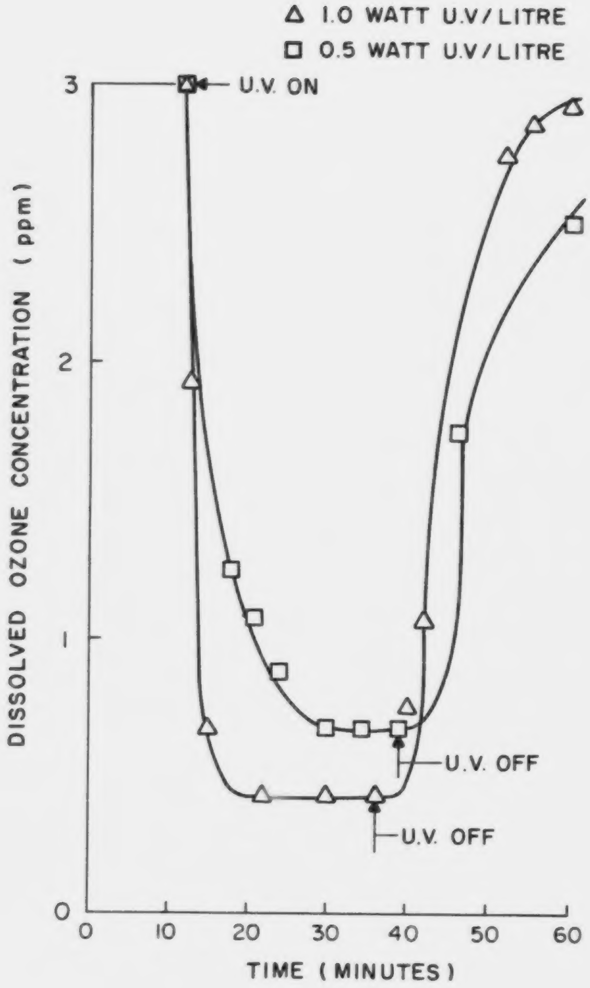


FIGURE 15 EFFECT OF U.V. ON OZONE CONCENTRATION AT 25°C - 10 g.hr^{-1} O_3 FLOW (CADOTTE 1978)

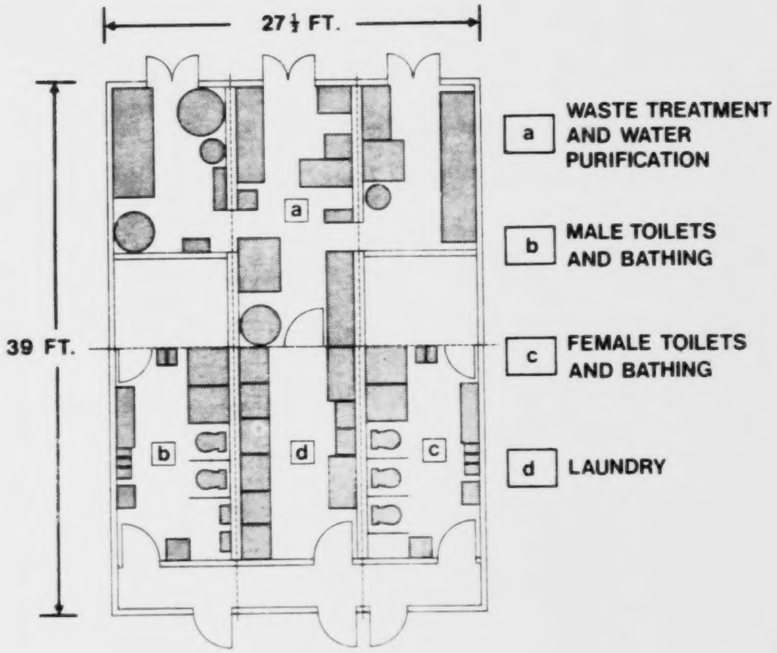

27½ FT.
39 FT.
a
b
c
d
a WASTE TREATMENT AND WATER PURIFICATION
b MALE TOILETS AND BATHING
c FEMALE TOILETS AND BATHING
d LAUNDRY

FIGURE 16. LAYOUT OF COMPLETE AIR-TRANSPORTABLE ESM.

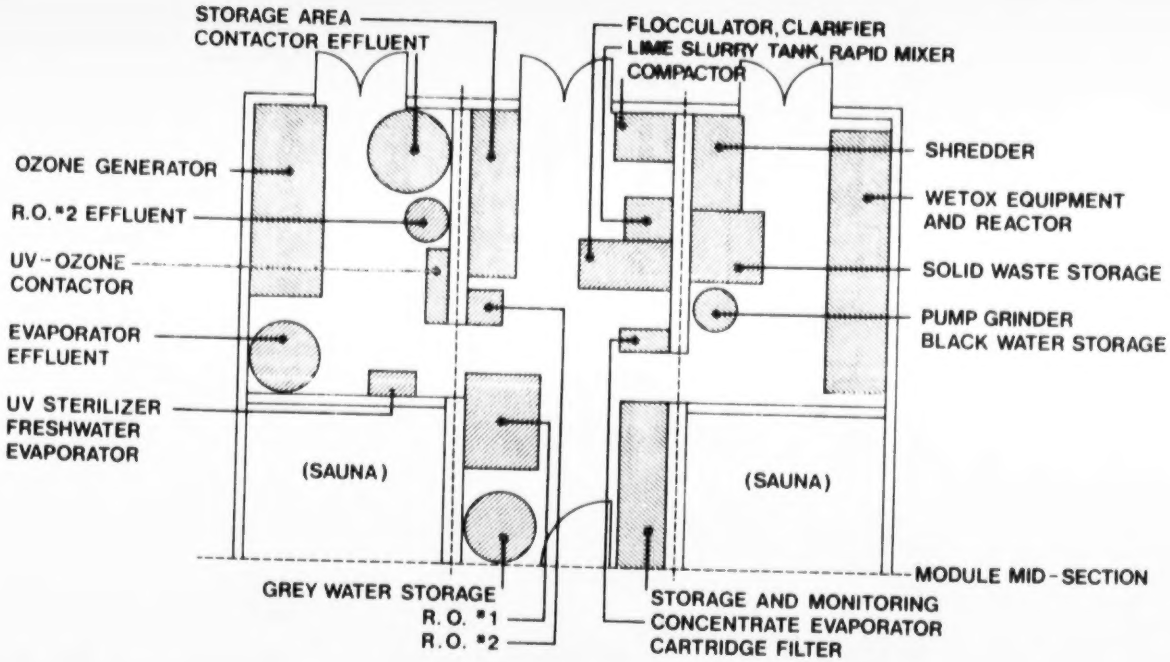


FIGURE 17. LAYOUT OF ESM PROCESS EQUIPMENT.

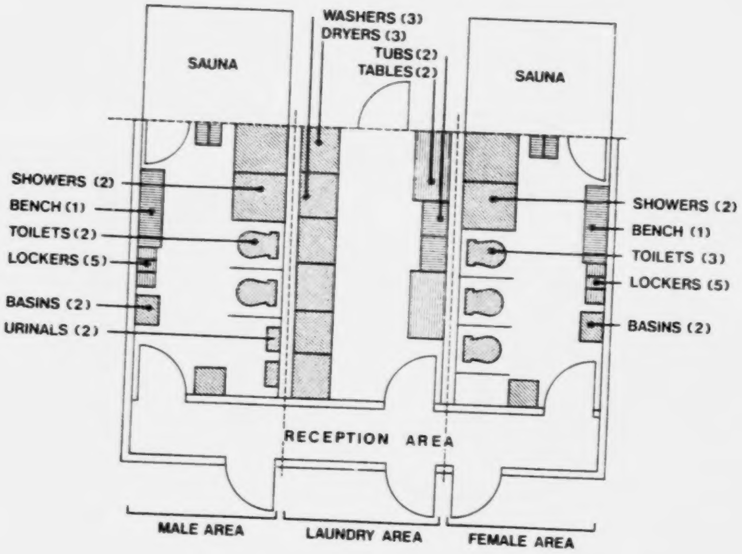


FIGURE 18. LAYOUT OF ESM PERIPHERAL FACILITIES.

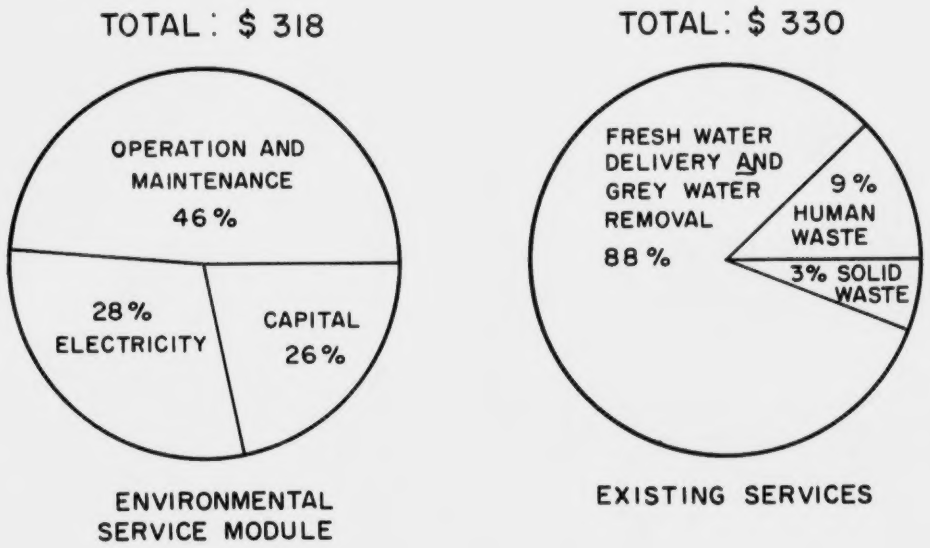


FIGURE 19. DAILY SERVICE COSTS FOR 150 PEOPLE

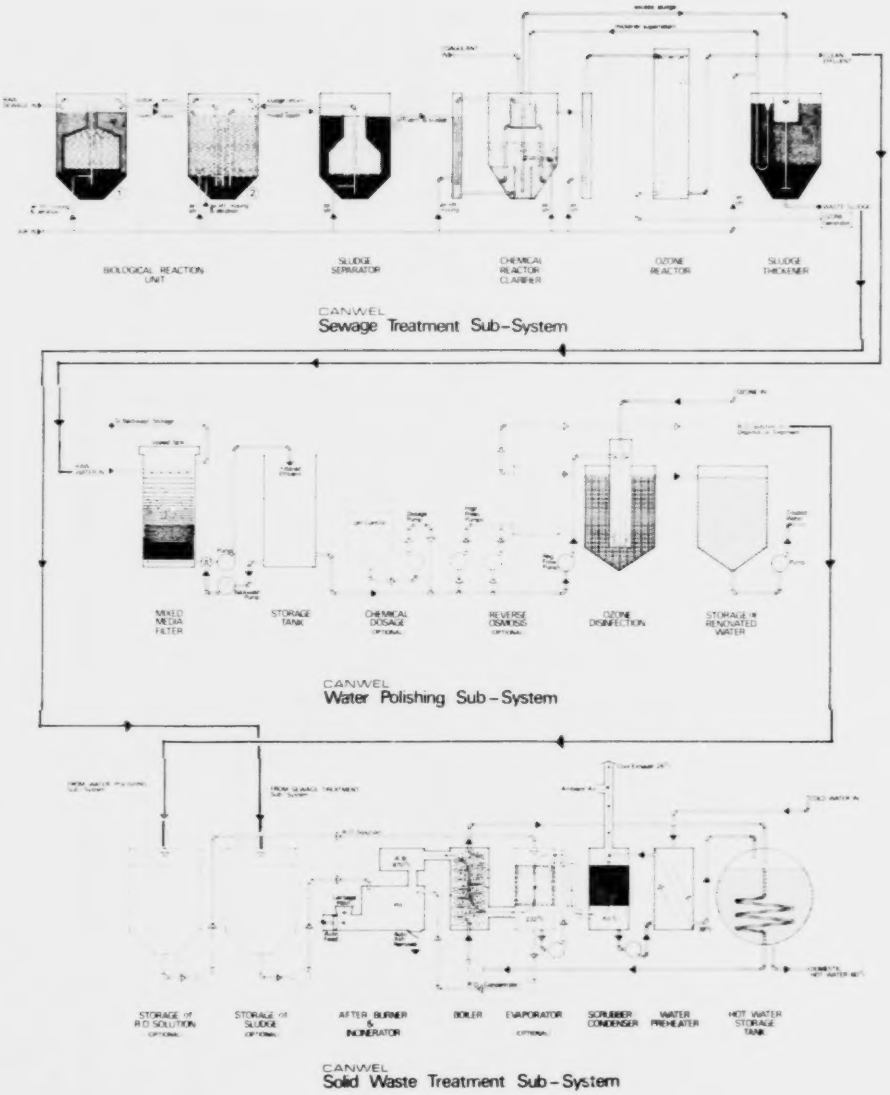


FIGURE 20. INTEGRATED CANWEL WASTE MANAGEMENT PROCESS.

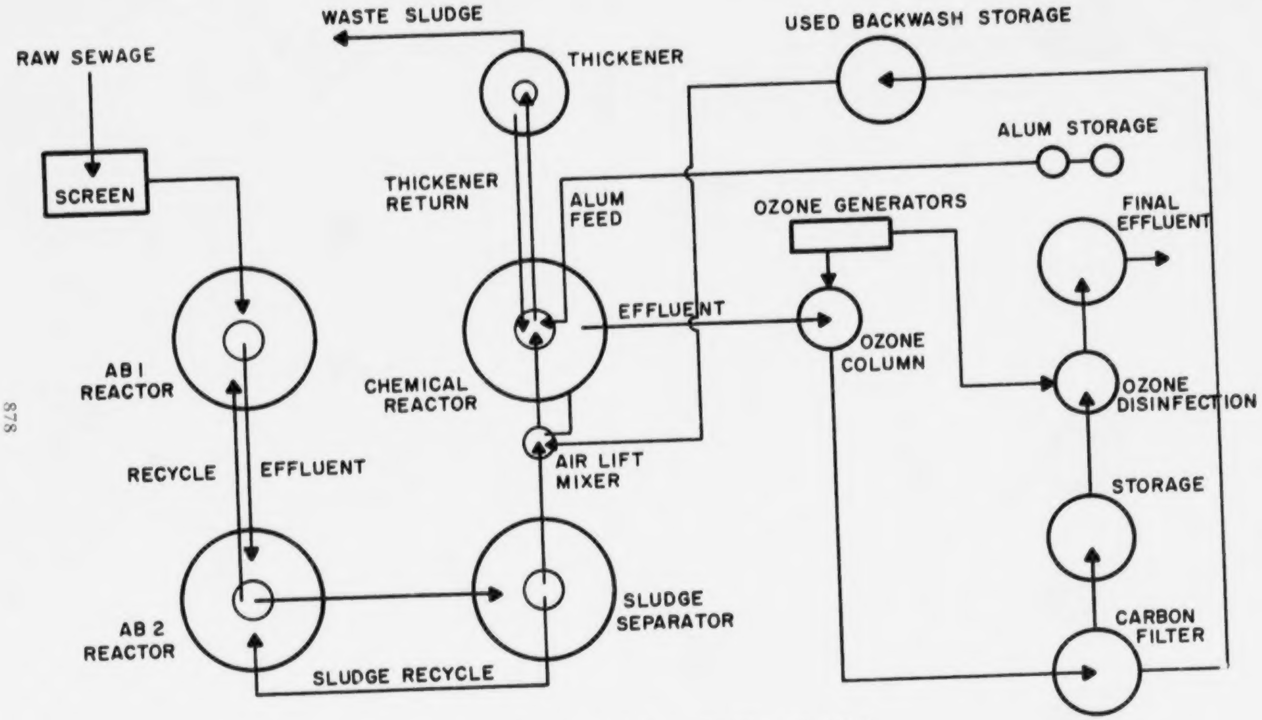


FIGURE 21. SCHEMATIC OF CANWEL PILOT PLANT.

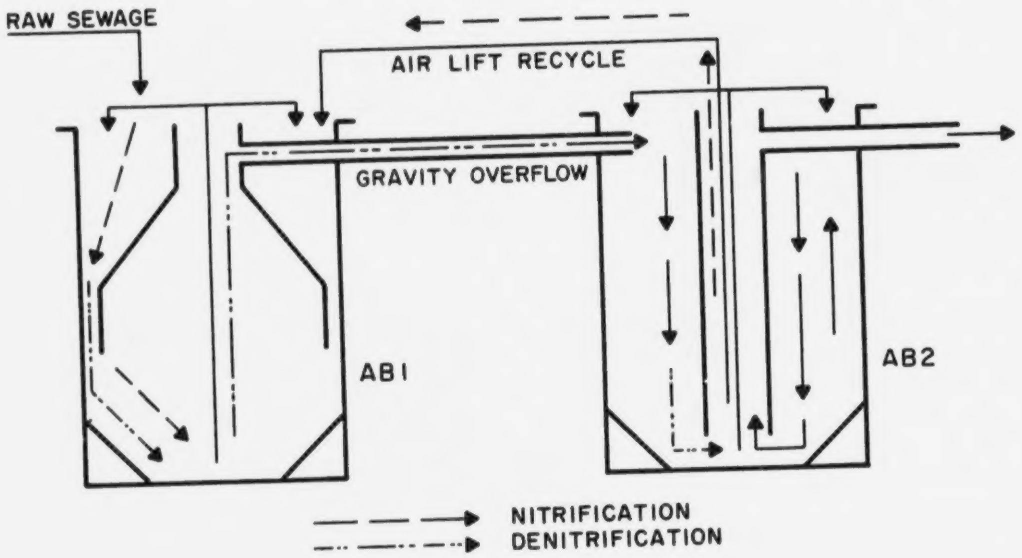


FIGURE 22 NITRIFICATION - DENITRIFICATION IN A CANWEL TWO STAGE COMPLETE MIX SYSTEM.

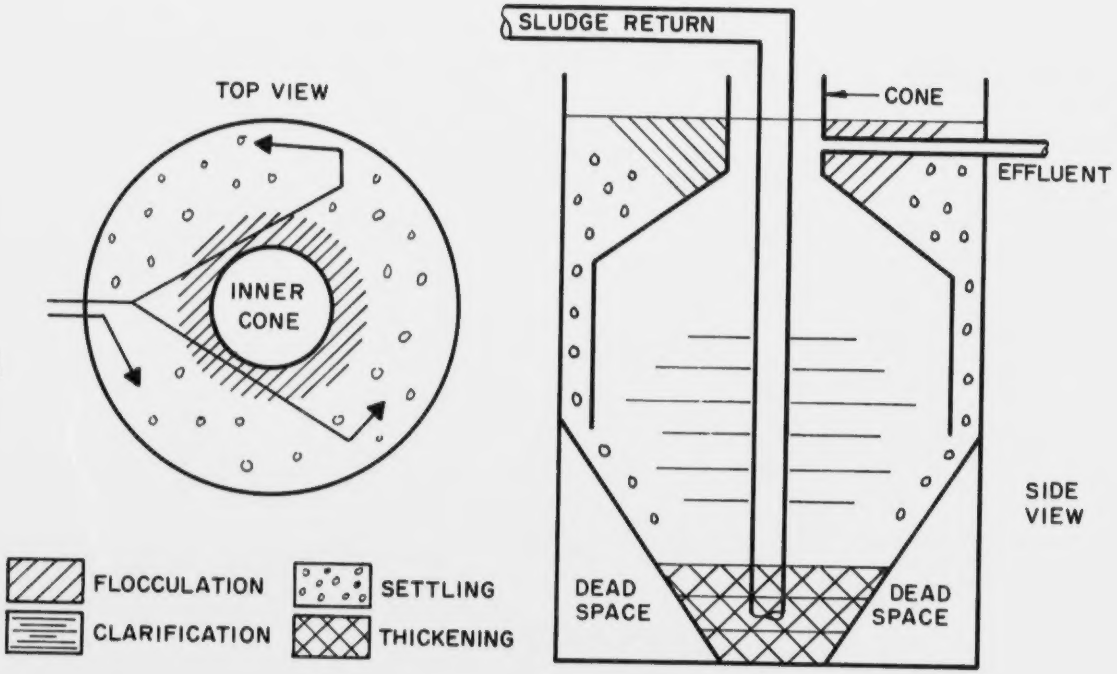


FIGURE 23. FLOCCULATION AND SETTLING IN THE CANWEL SLUDGE SEPARATOR

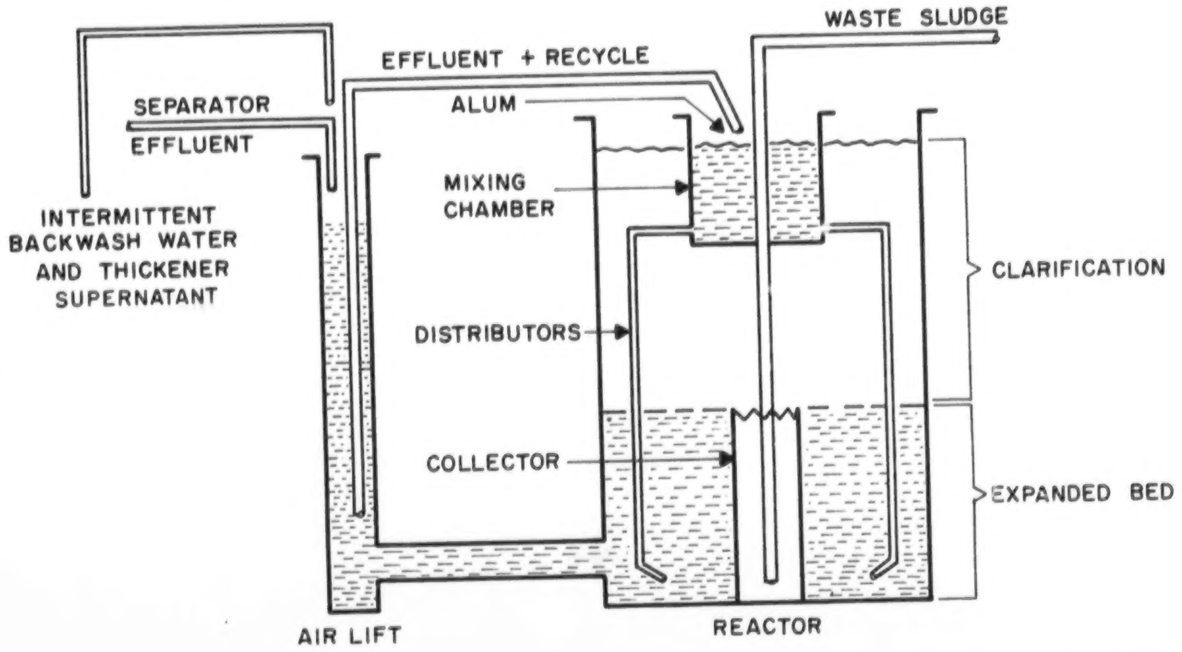


FIGURE 24. CANWEL CHEMICAL REACTOR CLARIFIER

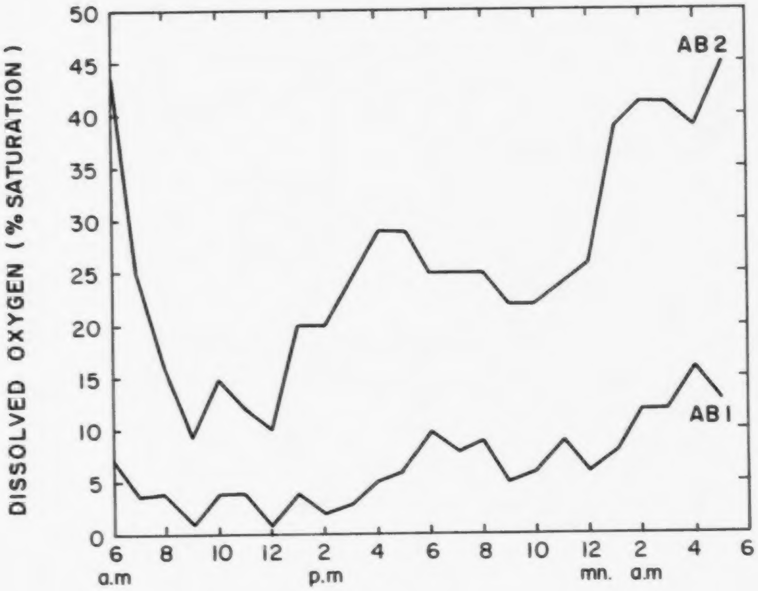


FIGURE 25. VARIATION IN DISSOLVED OXYGEN LEVELS IN CANWEL AB1 AND AB2 REACTORS OVER A 24HR. PERIOD.

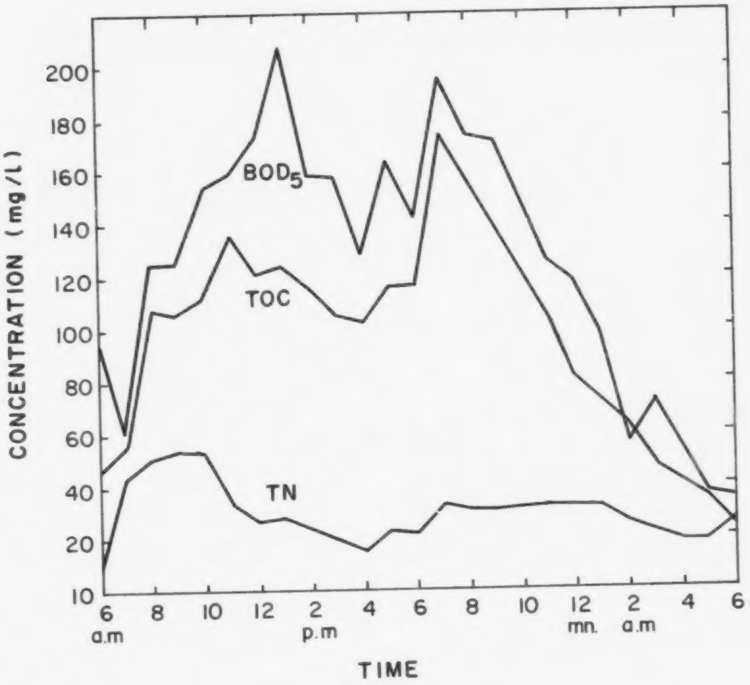


FIGURE 26. RAW SEWAGE 24 HOUR PROFILE.

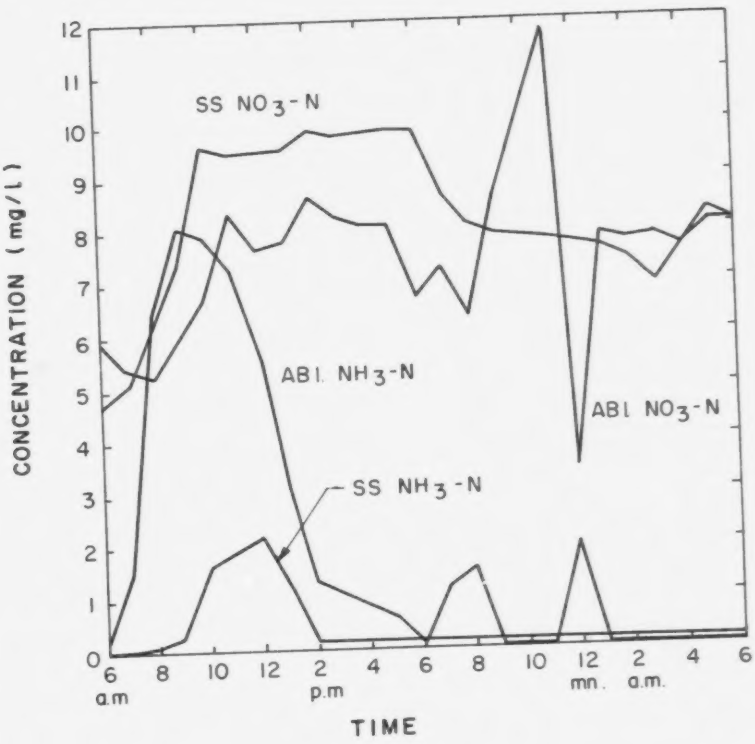
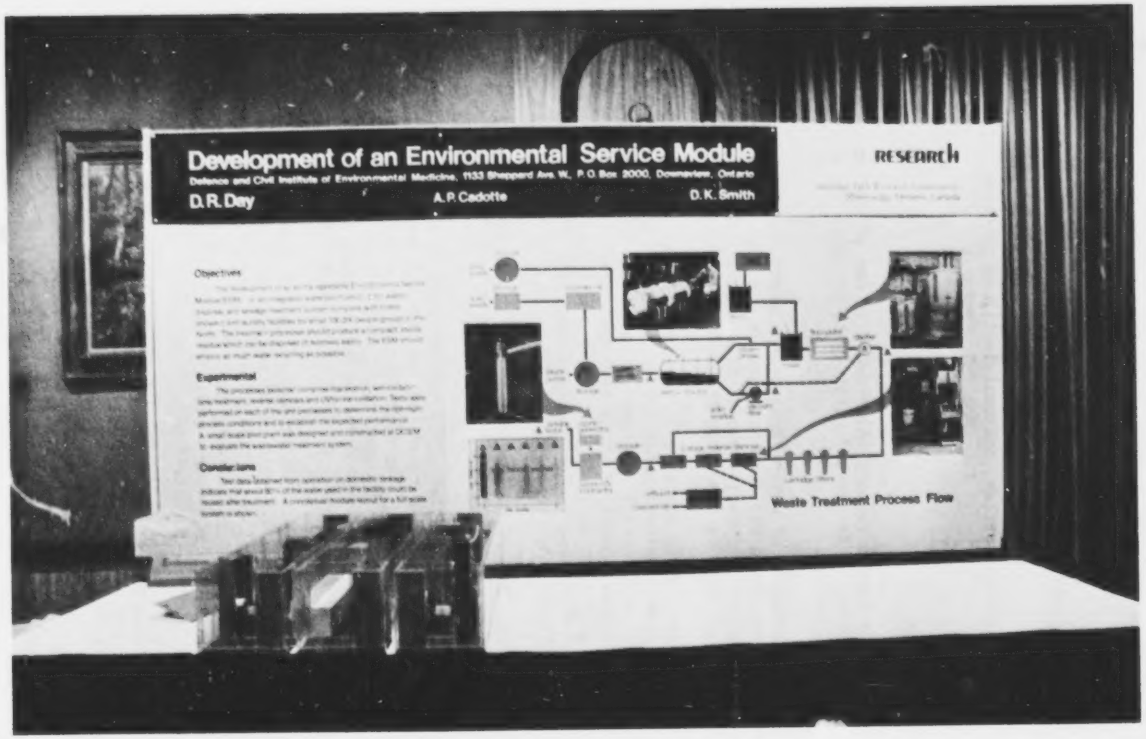


FIGURE 27. NITROGEN CONTENT OF CANWEL REACTOR EFFLUENTS OVER 24 HOUR PERIOD.


Development of an Environmental Service Module
Defence and Civil Institute of Environmental Medicine, 1133 Sheppard Ave. W., P.O. Box 2000, Downsview, Ontario
D.R. Day
A.P. Cadotte
D.K. Smith
RESEARCH
Objectives
Experimental
Conclusions
Waste Treatment Process Flow

REUSE OF WASTEWATER IN SOUTH AFRICA - RESEARCH AND APPLICATION

Peter E. Odendaal
Chief Adviser, Water Research Commission, P.O. Box 824, Pretoria

Lucas R. van Vuuren
Assistant Director, National Institute for Water Research,
P.O. Box 395, Pretoria

INTRODUCTION

In any water stress economy water reuse is one of the obvious first-line approaches in an overall strategy to optimize the utilization of available supplies.

The implementation of water reuse over a broad front is no simple matter, however: once reuse has been accepted as a policy objective, the responsible authorities will have to take special steps to create a climate which will be conducive to positive reuse decisions by consumers.

In this paper, South Africa will be considered as a case study of a country with an imminent water stress economy, where reuse has been recognized as an essential component of the national water management policy, and where positive steps have been taken to promote water reuse. Attention will also be given to present reuse applications, factors which are restrictive to more general reuse and the research programs aimed at alleviating these restrictions.

THE PROJECTED ROLE OF WATER REUSE IN THE SOUTH AFRICAN WATER ECONOMY

South Africa is a relatively dry country with an average annual rainfall of only 483 mm, compared to 735 mm for the USA and to a world average of 860 mm. In addition, the distribution of rainfall is uneven, varying from 50 mm p.a. on the west coast to 2 500 mm p.a. in the mountains of the south western and eastern regions. About a third of the country receives an annual rainfall of less than 300 mm (1).

Within the framework of present knowledge and expertise, it is being estimated that South Africa's total water credit - i.e. distributable water supplies - will be 33 000 million m^3 per year. Demand is rising rapidly and projections indicate that by the year 2 000 the demand will be 30 000 million m per year - only 3 000 million m^3 less than the total quantity of water available.

Measures could be applied, however, to reduce water demand and the following figures are based upon estimates by the Commission of Enquiry into Water Matters (2):

Savings through improved utilization of irrigation water	1 500 million m^3 p.a.
Savings through water reclamation and reuse	7 200 million m^3 p.a.
TOTAL REDUCTION IN DEMAND	8 700 million m^3 p.a.

It is important to note that the estimated 7 200 million m^3 p.a. which can be saved by reuse, constitutes almost 22% of the country's distributable water supplies. It is evident, therefore, that reuse is destined to play a key role in the South African water economy. This being the case, the question arises as to what policies, incentives and pressures exist in South Africa to promote water reuse.

PROMOTION OF REUSE THROUGH GOVERNMENT POLICY

Commission of Enquiry into Water Matters

In 1966 the State President appointed a Commission of Enquiry into Water Matters "to report upon and to submit recommendations on all aspects of water provision and utilization within the Republic of South Africa, as well as the broad planning of policy in this connection."

The Commission tabled its report in Parliament in 1970. With specific reference to reuse, the Commission pointed out that reuse will in future assume an increasing important role in augmenting the country's limited water resources and considered it essential that at least 50% of the gross water demand by municipalities serving large industrial centres, should be met by the reclamation of effluents. The Commission recommended that:

"(i) the Department of Water Affairs, in administering the Water Act (to be discussed further on), place the internal water use by industry and the reclamation of effluents for reuse on a practical footing, first, by issuing the necessary policy directive for the execution of a long-term water plan, and secondly, by defining the administrative procedures and instructions for the planning and erection of water reclamation schemes as integrated parts of the water economy of the areas to be served by them;

(ii) to promote sewage reclamation as an essential part of water supply on a national basis, consideration be given to subsidising water reclamation schemes for local authorities where consumption exceeds 1 137 m^3 d (250 000 gallons per day); and

(iii) adequate financial support be given to ensure that research work into water reclamation and the internal reuse of water in industry can be placed on a practical footing and substantially intensified."

As, with a few exceptions, the recommendations of the Commission were accepted by Government, and as these still constitute the guidelines for water planning in South Africa (1), water reuse is firmly entrenched in the country's official water management policy.

The South African Water Act (Act 54 of 1956)

In implementing its reuse policy, the Government has a powerful tool in the Water Act, which is being administered by the Department of Water Affairs. In fact, the promotion of water reuse was already implicit in the Act when it was promulgated in 1956.

The main features of the Act which have impact on water reuse are the following (3).

(1) For the use of more than 250 m^3 per day on average, or more than 300 m^3 on any one day, a permit is required from the Department of Water Affairs (Section 12 of the Act). In granting such permits, the Department has the authority to specify that reuse, recycling and hierarchical use of water should be practised as far as possible.

In certain instances, where water is in short supply, factories that wish to expand production may not be able to obtain permits for additional water supplies and are forced to use reclaimed water or to practise internal recycling and strict water conservation measures.

(2) A local authority may not construct, enlarge or alter a works in which more than 125 000 m^3 of water may be stored or by which more than 5 000 m^3 per day may be diverted, nor enlarge any other works to such an extent that more than the above quantities may be abstracted or stored, without a permit from the Department of Water Affairs (Section 13 of the Act). Through this control of water intake the Department has the power to exert pressure on local authorities to purify effluent for reuse where water is in short supply.

(3) Waste waters must be purified to standards laid down by the South African Bureau of Standards (Section 21 of the Act). Standards have thus far only been specified generally and for certain very pure headwaters, but may also be laid down for specified streams, for specified areas or for specified industrial purposes.

Where an industry discharges to a sewer owned by a local authority, it does not need a permit, as the responsibility then rests with the local authority to meet the general standards. The Department may also grant permits of exemption in cases where meeting the standards would be unduly onerous, in which instances special conditions are laid down.

While the primary intention of this clause is to control pollution, it does effectively encourage the reuse of wastewater: where industries and local authorities are forced to spend money on meeting the effluent standards, it becomes likely that they will investigate opportunities for reuse or may even be prepared to further upgrade effluents for this purpose.

(4) Water abstracted from a stream and used for industrial or municipal purposes must be returned to the stream of origin after purification. Before return, the water must be diminished only be essential consumptive use. For instance, evaporation of a tractable waste water to avoid the expense of purification, or irrigation of inferior land where there is a possibly better form of reuse, would not be acceptable (Section 21 of the Act). This requirement is intended to ensure the indirect reuse of wastewaters.

(5) Section 22 of the Act makes provision for local authorities to use purified sewage effluent for any approved purpose or to discharge it to a public stream. This provision was designed to overcome older legislation which forbade the discharge of effluent to public streams and also opens the way for the reuse of sewage effluent.

(6) Shortly after the Commission of Enquiry submitted its report in 1970, the definition of a 'waterworks' in the Water Act was amended to include a 'sewage' works. Prior to the amendment, the Department of Water Affairs would only grant loans and subsidies for the construction of water works and not sewage purification plants. The amendment cleared the way for the Department to use its discretion in the granting of subsidies or loans for promoting the construction of water reclamation plants.

The implementation of these sections of the Water Act has already resul=

ted in various reuse applications. Research is also forging ahead and as new knowledge and technology become available, the scope for reuse is steadily increasing. In the following section the present status of water reuse in South Africa is reviewed and this is followed by a discussion of the problems which stand in the way of more general reuse and of how the South African research programmes are geared to these problems.

RECLAMATION OF PURIFIED EFFLUENTS IN SOUTH AFRICA

Table 1 indicates the total quantity of purified sewage effluent produced by 33 major cities, towns and industrial complexes in South Africa, and the quantities of effluent reused for irrigation, power plant cooling and industrial use (4).

Included in these figures are those for the Pretoria-Witwatersrand-Vereeniging complex (PWV-complex), the most densely populated and industrialized area in the country, which produces almost 50 per cent of South Africa's gross national product. This area is under mounting pressure as far as water supply is concerned, and it is significant to extract the figures applicable to this area and compare them with those for the remaining areas included in Table 1. This comparison is drawn in Table 2.

The much higher reuse percentages for the PWV-complex are indicative of especially the quantitative control on water supplies, in terms of the Water Act, which are operative in the area.

The categories in which purified sewage effluents are at present being reused in South Africa are the following:

Irrigation

In South Africa the policy relating to the reuse of effluents for irrigation, differs in many respects from the policies being followed in some other countries. The South African approach is that in general the irrigation of effluents should be discouraged, with the following two exceptions (3):

1 The irrigation of public parks, and sportsfields which would otherwise require fresh water.
2 Near the sea, where the wastewater would otherwise be discharged to sea and where a better use for the water cannot be found.

The fact that such large quantities of purified effluent are still being used for irrigation, e.g. 26,8% in the PWV-complex, is mainly due to circumstances which prevailed before the promulgation of the Water Act in 1956, when local authorities had no other option but to resort to this means of disposal. Since the introduction of the Water Act the percentages of effluent used for irrigation have steadily dropped through the diversion of purified effluents to the streams of origin or for other uses.

Apart from the instances 1 and 2 mentioned above, the reuse of effluents for irrigation is as far as possible limited to cases where the producers of effluent find it difficult to meet the effluent standards for river discharge. In such cases, irrigation is primarily regarded as a means of disposal rather than reuse, and permits are issued only for a limited period of time to give the user an opportunity to comply with the standards. Before granting such a permit, the Deparment of Water

Affairs consults the Soil and Irrigation Research Institute of the Department of Agricultural Technical Services on the feasibility of the project.

At present about 240 permit holders are permitted to irrigate 150 million m^3 of effluent per annum - see Table 3 (5). Assuming an application of 500 mm p.a., the total volume of effluents used for irrigation in South Africa is sufficient for an area of about 31 000 ha, which is about 3% of the total area under irrigation.

Most of the irrigated effluents - about 85 per cent - are produced by municipalities. Municipal effluents are used for the irrigation of pastures, plantations, golf courses, sportsfields and parks. Permit holders other than municipalities generally use the effluents to irrigate perennial crops having irrigation requirements, such as pastures, fodder, sugar cane and trees, thereby limiting overirrigation and reducing the land area as far as possible. These effluents generally do not meet the TDS and COD standards for river discharge.

The South African Department of Health formulated guidelines with respect to the public health aspects of irrigation with sewage effluent. In general, no irrigation with sewage effluent is allowed on crops that may be eaten raw. Surface irrigation of fruit trees, trellised vines and crops not eaten raw is allowed with effluents that have undergone at least tertiary treatment and contain less than 1 000 E. coli type 1 per 100 cm^3. The same quality requirements apply to irrigation of pastures for beef cattle, provided no grazing is permitted during irrigation. Dairy cattle may not graze on these pastures. Similarly, recreation facilities such as golf courses, parks and sportsfields may be irrigated when not in use. There are no restrictions on the quality of effluent for irrigating crops from which dry fodder is to be produced, providing contact by humans and animals is prevented (6).

Two interesting irrigation projects which should be mentioned are being operated by the Johannesburg Municipality and AECI (chemicals and explosives manufacturer) respectively:

(1) Johannesburg Municipality. The Johannesburg Municipality has been irrigating pastures with sewage effluent since 1914 (7). Today the scale of operation involves 3 200 breeding cows and a total of 7 500 head, making it the largest intensive breeding enterprise on irrigated pastures in South Africa. The total area under irrigation is 1 600 ha, the total volume of sewage effluent being irrigated is 113 500 $m^3 d^{-1}$ and the application rate is 230 cm per annum. At this rather high application rate, between 40 and 50% of the effluent applied find its way back to the river, particularly during wet weather.

(2) AECI Ltd. The Modderfontein factory of this company produces explosives, fertilizers and industrial chemicals such as sulphuric acid, nitric acid, ammonium nitrate, methanol and formaldehyde. One of the factory effluent streams contains nitrogen at a concentration of about 3 000 mg/l and is sprayed onto grassland at a rate of 466 kg N/ha per annum. The project commenced in 1963 and covers an area of 857 ha. The grass, sown and natural, is either grazed by cattle or made into hay (8). As the total ammount of liquid applied to the soil per annum is only about 38 mm (1,5 inches), the scheme is not one of irrigation but rather the application of fertilizer to grassland.

Power Plant Cooling

About 67% of the total demand for industrial water in South Africa, the mining industry excluded, is required for cooling, with power stations being the largest consumers for this purpose (9). Evaporative losses at power stations are high and amount to roughly 1 gallon (4,5 l) for every kWh generated (10).

Purified sewage effluents have been used for cooling at municipal power stations in South Africa for many years - Johannesburg's Orlando power station, for example, since its commissioning in 1942 (11). The extent to which sewage effluent is used for power plant cooling in South Africa, is shown in Table 4 (12).

Operational experience gained over a period of 40 years has shown that under controlled conditions no serious corrosion or fouling problems are likely to occur when using purified sewage effluent in power station cooling circuits. The blow-down must be regulated however, in order to comply with the effluent discharge requirements of the Water Act, which limits the extent to which the cooling water can be recirculated.

Pulp and Paper Industry

Practical experience over the past years at two large South African pulp and paper mills and at one board mill has proved that purified sewage effluent can be used as process water without restrictions, even in the production of fine paper (12).

The Enstra Mill of South African Pulp and Paper Industries Ltd. (SAPPI), near the city of Springs, was the first large manufacturing industry in the country to reclaim sewage effluent as the major part of its water supply. The present supply consists of 16 000 m^3 d^{-1} potable water from the Rand Water Board and 27 000 m^3 d^{-1} reclaimed from secondary effluent provided by the Springs Municipality (4).

Initially the secondary effluent received only limited tertiary treatment consisting of sand filtration and low-level chlorination. The demand for a higher brightness paper called for further upgrading of the effluent, and based on investigations by the staff of the factory and the National Institute for Water Research (NIWR), a full-scale advanced treatment plant was commissioned in 1970. The treatment consists essentially of flocculation with aluminium sulphate and polyelectrolyte, flotation, pH adjustment with sodium hydroxide, chlorination to prevent algal growth, and sand filtration.

The quality of the reclaimed water allowed its successful use in all sections of the mill, and the brightness of the paper produced actually exceeded that of paper produced with Rand Water Board water. A cost saving of more than 50% in comparison to the conventional water supply is achieved and the annual saving is of the same order as the initial capital expenditure on the reclamation plant.

The Mondi Paper Company near Durban is another paper mill that reclaims sewage effluent for process water. The mill, with a production capacity of approximately 750 t d^{-1} of newsprint and fine paper grades, was dedigned to use up to 12 000 m^3 d^{-1} of reclaimed sewage effluent from the nearby Durban southern sewage works as process water (13). One of the conditions imposed by the Durban Corporation for the sale of the land on which the mill was to be erected, was the use of a minimum of 9 000 m^3 d^{-1} of treated sewage effluent as process water. Pilot plant studies by the staff of Mondi and the consulting engineers in 1969, showed that the tertiary treatment of sewage effluent could produce reclaimed water of a quality that was within the limits of tolerance for all paper grades to be manufactured by the mill.

Since the start-up of the mill in 1971/72, the quality of the reclaimed water has been such that no difficulties have been experienced in production. During a 29-day emergency period in 1976, when a number of industries in Durban had to be shut down because the potable water supply system had been damaged by floods, the production of the mill was kept at full capacity with reclaimed effluent as the only supply. Experience then, and during other periods when the supply of potable water was less than 10% of the total mill demand, showed that the reclamation plant always produced water which met the quality requirements for newsprint and fine paper production. The treatment system consists of chemical flocculation, foam fractionation, sand filtration, and activated carbon treatment, for which the carbon is generated on site.

In this instance the cost of the reclaimed water is of the same order as that of the conventional potable water supply (just over 16c per m^3 in 1976), so that there is actually no cost incentive for the mill to use reclaimed water. The point emerges, therefore, that if industries are to use sewage effluents, it will be necessary to maintain the basic charge at a level which will keep the total cost of reclaimed water below that of the conventional potable supply.

At S.A. Board Mills, Bellville, purified sewage effluent constitutes about two thirds of the total water intake. The mill produces lined grey backed or manila backed board, either coated or uncoated, at a rate of 71 t d^{-1}, using 570 m^3 d^{-1} of potable water and 952 m^3 d^{-1} of purified sewage effluent (12).

Other Industries

Some of the other notable instances where reclaimed effluents are being used by industries are the following (4):

(1) The Portland Cement factories at Slurry and Ulco together use 2 200 m^3 d^{-1} of purified sewage effluent in their slurry processes. The effluent need not be of a high quality for this purpose as all the water and organic matter are volatilized in the rotary kilns.

(2) Dunswart Iron and Steel Ltd., Benoni, producing 18 000 t per month of rolled steel sections from scrap, pig iron and ore, uses about 1 700 to 1 800 m^3 d^{-1} of purified sewage effluent and about 700 m^3 d^{-1} of potable water. The purified sewage effluent from the City of Benoni is employed mainly for cooling of the continuous casting plant, and partly for washing. Dunswart is expanding and the company has applied for permission to increase its use of purified sewage effluent to about 3 300 m^3 d^{-1}.

(3) 1 100 m^3 d^{-1} of treated sewage is used for grit washing at the De Beers diamond mines, Kimberley.

(4) Sasol (oil from coal industry), Sasolburg, uses 23 000 m^3 d^{-1} of purified sewage effluent for the conveyance of ash from the power station, followed by maturation pond treatment and subsequent discharge into the Vaal River.

(5) The gold mines in the Welkom area use 13 200 m^3 d^{-1} of purified

effluent from slurry and conveyance.

An interesting possibility which is being considered, is to use saline mine and industrial effluent to pump more than 1 million tons of coal per month, some 600 kilometres from the interior to the coast. This will not only abate some severe pollution problems, but will free rail transport for other uses and enhance the export potential of South Africa's newest harbout at Richard's Bay (3).

INTERNAL REUSE OF WATER BY INDUSTRY

Under the pressures of limited water supplies, several South African industries have adopted intensive water conservation and recycling practices. The extent to which such practices can be effected by reductions in water supply, was illustrated in the Vaal Triangle when a severe drought in 1966 forced water cuts of 25% and later 30%. A saving of 27% of the estimated unrestricted demand was achieved, and although restrictions were lifted in February 1967, consumption in 1968 was lower than in 1965 and totalled only 82% of the anticipated demand (14). This limited the scope for further savings, however, and when it again became necessary to impose restrictions, in 1969, the aim of a 25% saving could not be realized and a reduction of only 9% on estimated normal requirements was achieved.

The extent of water recycling in four of South Africa's largest industries are summarized in Table 5 (4).

A short review of the systems in each of these industries are given:

Iscor Steel Mills (12)

The Iscor Steel Mills at Pretoria and at Vanderbijlpark both operate on a water quality cascading system, where the best quality water is used where there is a definite need for it, and the bleed-off or blow-down from one system feeds the next system and so on.

Steel production at the Pretoria works started in 1934 and since water is relatively scarce, the reuse of water was already planned at the design stage, but further improvements to the reuse systems have been made over the years. The following figures give an outline of water recirculating rates and consumption, for a daily production of 4 200 t of steel (in $m^3 d^{-1}$):

Daily water intake	21 300
Daily circulating rate	730 000
Water circulated per ton of steel	175
Water consumed per ton of steel (evaporation plus effluent)	5,1
Water discharged as effluent	4 100

Thermal Power Stations (12, 15)

The South African Electricity Supply Commission (ESCOM) operates 19 thermal coal-fired power stations with a capacity of 13 400 MW. Two large stations with 3 600 MW rated capacity are being constructed at present. Twelve of the power stations are situated in the Highveld of the Eastern Transvaal and Northern Orange Free State and draw their water from the Vaal River or its tributaries.

In 1965 restrictions were placed on the use of water from the Vaal River and cooling water circuits were then operated with a reduced blow-down. The recycling wet cooling systems being used by ESCOM are summarized in Table 6.

In the case of the blow-down only system there is no treatment, and the number of cycles is limited by the specification of the general standard, as laid down in terms of the Water Act, that TDS should not be increased by more than 500 mg/l above intake water. Acid dosing can be done with H_2SO_4 or with acid mine drainage - e.g. at the Wilge power station 50 000 to 60 000 m^3 d^{-1} of acid mine water with pH 2 is received in the cooling water system. The limiting factors are the concentrations of sulphate, magnesium and silicate, and these limitations also apply when preclarification is added to the system.

During 1968 many schemes were examined to render excess ash handling water more suitable for reuse or for direct discharge to streams. Even=tually it was realized that through integration of the ashing and cool=ing systems, C_aCO_3, $Mg(OH)_2$, and silicate could be precipitated. As a result, concentration cycles of up to 50 times are possible with waters in the Eastern Transvaal.

The extent to which ESCOM implemented recycling in its cooling systems, is reflected in Table 7.

South African Coal, Oil & Gas Corporation (Sasol)

Sasol 1, the largest and the only commercially viable oil-from-coal plant in the world, has been in production since 1955. In addition to petrol, the plant produces virtually the full range of products derived from crude oil, as well as a number of others that are manufactured in satellite petro-chemical plants.

The factory has a daily water intake of 63 600 m^3 and returns on average 31 500 m^3 to the Vaal River (16). Through intensive recycling this volume of water performs functions which would otherwise require 823 000 m^3. Sasol 2, which will be a much bigger plant, is now under construc=tion, and the water circuits have been designed to operate on a closed loop system.

African Explosives and Chemical Industries (AECI), Modderfontein (4, 17)

This is one of South Africa's largest industrial concerns and also the largest dynamite factory in the world. The Modderfontein factory had a potable water consumption of approximately 16 500 m^3 d^{-1} in 1968. These supplies are limited by permit and on two occasions since 1966 these have been reduced during drought periods. Intensive recycling and reuse of process waters considerably reduced the factory's fresh water intake to 14 000 m^3 d^{-1} in 1973 in spite of considerable expan=sions in production.

An important point that should be made in relation to the instances of effluent reclamation or internal recycling that were cited in this and the previous section, is that in most cases the municipalities and in=dustries involved had the technical staff who in the first instance undertook the investigations and research which led to the particular reuse applications, either independently or in collaboration with a re=search agency, and secondly were competent enough to oversee the sub=sequent operation of the reuse systems. This availability of competent individuals is a basic requirement for the broad implementation of water reuse systems.

RECLAMATION FOR POTALBE USE

The well-known Windhoek reclamation plant is still the only full-scale facility in the world where sewage effluent is directly reclaimed for potable use. Windhoek is the capital of South West Africa, a territory which is being administered by South Africa under a mandate from the old League of Nations. Although the city, therefore, is not in the Republic of South Africa, the reclamation plant resulted from research which the NIWR in Pretoria had done in collaboration with the City Engineer's Department in Windhoek.

In spite of the fact that the original Windhoek plant had already been commissioned in 1969, no water reclamation schemes for potable water have subsequently been established in South Africa. The reason is simply one of relative urgency. Windhoek is situated in a very arid region, and at the time when the reclamation plant was commissioned, the city could no longer meet its water demands from the available conventional supplies. In South Africa no city has yet reached the same state of urgency, but in a few areas the economically feasible alternatives are rapidly being exhausted and South Africa's first operational reclamation plants for potable water can be expected to appear during the late eighties.

In this realization, South African research into water reclamation for unrestricted reuse has continued, and in fact intensified, since completion of the Windhoek project. It should be realized that the situation in Windhoek in two respects uniquely facilitated the introduction of reclamation: In the first instance, industrial effluents could be easily segregated, leaving an essentially domestic effluent for reclamation; secondly, the particular siting of the sewage purification facilities allowed the reclamation plant to be integrated with the conventional water purification plant, which meant considerable savings in pumping and piping and enabled the same staff to operate both plants. For this reason the Windhoek scheme could not serve as a blueprint for potable water reclamation in South Africa, where a high level of industrialization has been achieved and where the planning of reclamation schemes will have to contend with a status quo where sewage purification facilities have not been sited and planned with future reclamation in mind. This situation places a number of restrictions on expanded reuse, and in the following section the main problems are discussed, with specific reference to ongoing research programs directed at solving these problems.

PROBLEM ORIENTATED RESEARCH

One of the recommendations of the Commission of Enquiry into Water Matters, mentioned previously, was that water research in general should be escalated and co-ordinated to a greater extent. It also made a specific recommendation that research into the reuse of wastewaters should be intensified. With the establishment of the Water Research Commission in terms of the Water Research Act (Act 34 of 1971) in 1971, the machinery was created to implement these recommendations.

The Commission is charged to co-ordinate water research, to accelerate water research through selective funding and to promote the dissemination and application of research findings and technology. The Commission which derives its funds from a special levy on all water consumption, does not carry out any research by itself, but contracts research agencies for this purpose. Since its establishment the Commission has given special attention to the funding of research contracts on water reclamation in order to supplement the programs being carried out by the National Institute for Water Research in this field.

South African research relating to water reclamation has mainly been associated with the Windhoek plant and with four pilot plants, two in Pretoria and two in Cape Town, as indicated in Table 8. The main problems receiving attention are discussed below.

Health aspects

If the large-scale reclamation of sewage effluent for potable use is to become a reality in South Africa, the Department of Health with whom the final clearance will rest, must obviously be satisfied that health requirements will be met. A strong research effort is, therefore, being maintained to provide the scientific data which can guide the Department in its decision making. The lines of investigation have been the following:

(1) Monitoring of Plant Performances. One of the conditions on which the health authorities approved the direct reclamation of sewage effluent for potable use in Windhoek, was that a strict quality surveillance program should be maintained. Virological surveillance is being done independantly by the South African Institute for Medical Research (SAIMR) and the NIWR; bacteriological surveillance by the SAIMR, NIWR, Windhoek Municipality and the Department of Water Affairs. The results of the different laboratories generally correlated well and thus far the quality of the final water has always conformed to standards for drinking water (USPHS, WHO and the South African Bureau of Standards). Viruses have not been isolated from any final water sample.

The SAIMR, NIWR, National Institute for Virology, Rand Water Board, and the Pretoria Municipality are all collaborating in the bacteriological and virological monitoring of the Stander reclamation plant in Pretoria. In this case too, the final water consistently conformed to international standards for public water supplies.

(2) Epidemiological Surveys. Epidemiological studies by the SAIMR have now been proceeding for a number of years in the Windhoek municipal area. Thus far the studies have indicated that disease patterns in the area show no relation to the quality of the water supply. Data will be accumulated for several years and computer stored for analysis.

Projections indicate that water reclamation for potable use, may well have to be introduced in the Cape Town area in the late eighties. For this reason, it is considered a matter of urgency that epidemiological studies commence as soon as possible to establish health patterns for communities that may receive reclaimed water. Comparative data would be required for such communities before and after the introduction of reuse, and comparisons should also be drawn with other communities that are supplied from conventional sources only. Negotiations are now proceeding whereby such studies will be undertaken by the Department of Community Health at the University of Cape Town.

(3) The Occurrence and Fate of Micro-pollutants. A twofold study was carried out by the NIWR on the occurrence of organics in the feed and final water of the Stander plant and on the efficacy of a

pilot plant to remove inoculated toxic organic compounds (18).

In the feed to the Stander plant, a total of ten polynuclear aromatic hydrocarbons were detected in the ng l^{-1} range, whilst the final water contained only two of these compounds (pyrene and fluoranthene), neither of which have toxicological importance. The percentage reduction of organic material through the plant was 84%. The total number of chromatogram peaks in the final water was almost the same as in the feed water, but a marked change had taken place in the distribution of the peaks. The final water mainly contained peaks (more than 61%) in the concentration range 10 - 100 ng l^{-1}, whereas the peaks in the feed water were mostly (65%) in the range 100 - 1 000 ng l^{-1}.

Eighteen known toxic compounds were inoculated in rather large concentrations (up to 4,3 mg l^{-1}) into the feed water of the pilot reclamation plant. The plant reduced the toxicants by more than 99,9% to values ranging from 0 to 6 μg l^{-1} in the final water.

(4) Toxicological Studies. In an experiment which was run over a period of 25 months, the National Research Institute for Nutritional Diseases of the South African Medical Research Council and the NIWR subjected four different waters (reclaimed water from the Stander plant, humus tank effluent, distilled water, and tap water) to biological assaying, using rats (19). In addition spent and new active carbon were also included in the diets of the rats. During the experimental period a careful note of the health behaviour and sexual patterns of the rats was kept. All animals were autopsied and the liver, kidneys, lungs, heart, spleen, pancreas, gonads, oesophagus, stomach, intestine and any other abnormal tissues were fixed, stained and microscopically examined. Results indicate that the feed water to the reclamation plant (humus tank effluent) was the only water which exerted obvious deleterious effects on the test animals. No indication of the presence of carcinogens in reclaimed water was detected in this study. The lifelong ingestion of spent active carbon appeared to be without ill effects on the health of the rats, which substantiated results achieved in an earlier study (20).

The Ames Samonella auxotroph test for the detection of mutagenic substances in water is now being applied by the NIWR (21).

(5) Development of Biomonitoring Systems. Research by the NIWR on the development of biomonitoring systems which can be used at water reclamation and conventional purification plants, has thus far concentrated on (21):

- The respiratory performance of the protozoan Tetrahymena pyriformis in the presence of toxicants.
- The effect of toxicants on fish movement patterns and breathing rates.

(6) Monitoring the Chemical and Biological Quality of Water Supplies. On behalf of the South African Department of Health, the NIWR is maintaining a monitoring program on the chemical and microbiological quality of various waters, including conventional drinking-water supplies, reclaimed water, purified effluents, rivers, dams and boreholes in the PWV-complex (21).

A list of 11 compounds classified by an expert committee of WHO (22) as being very toxic and frequently present in water, was modified from various publications, and adapted to South African conditions by the NIWR (23). This list includes the trihalomethanes and other organohalogens, chlorophenols and chlorinated and polynuclear hydrocarbons. Based on toxicity data and limits set elsewhere (e.g. by the US EPA), limits of detection were established: 1 $\mu g\ l^{-1}$ for the chlorophenols and volatile organohalogens, and 0,1 $\mu g\ l^{-1}$ for the others.

Results indicated that the waters were generally of excellent quality. No polynuclear aromatics occurred above the minimum limits, with only traces of fluoranthene (<0,05 $\mu g\ l^{-1}$). Maximum values for organohalogens were 40 $\mu g\ l^{-1}$, which is below the limit of 100 $\mu g\ l^{-1}$ suggested by EPA. Chlorophenols were found in the range 1 - 10 $\mu g\ l^{-1}$. Only one water source (Hartebeespoort Dam) receiving heavy inflows of secondary effluent and serving several small communities near Pretoria, contains chlorinated hydrocarbons in the range 0,1 to 1 $\mu g\ l^{-1}$ (EPA limit is 4 $\mu g\ l^{-1}$).

Segregation of Industrial Effluents

In view of the world-wide controversy over the presence and possible deleterious effects of certain organic compounds in water supplies, it seems only logical to have a closer look at the sources of these compounds. It is likely that in most instances refactory organics will originate from a few point sources only. If these could be traced and eliminated at source by separate treatment or containment, it would make a tremendous contribution towards safeguarding the quality of receiving waters which serve as raw water supply for conventional purification plants. In cases where the direct reclamation of sewage effluent for potable use is under consideration, it would also remove much of the misgivings - whether justified or not - which may still exist about health aspects. The application of this philosophy was in fact a key factor which facilitated the introduction of reclamation for potable use in Windhoek, in that industrial effluents were segregated from the 'catchment' for the reclamation plant.

As mentioned previously, it is likely that Cape Town will have to reclaim water for potable use in the late eighties. In its planning for the future, the city has already taken steps to segregate industrial effluents from the sewage flow which could eventually be reclaimed for potable use. Two pilot plants are being operated in the area: one at the Athlone sewage works which receives most of the industrial effluents, and where investigations are carried out on the reclamation of the effluent for industrial use; the other pilot plant is on the Cape Flats where a new sewage works is under construction which will treat predominantly domestic sewage. The Cape Flats pilot plant is used to investigate reclamation for unrestricted reuse.

The segregation of industrial effluents as achieved in Windhoek and Cape Town may not always be feasible in other situations. Should reclamation for potable use be contemplated in such instances, catchment quality control should be practised for the sewerage networks involved, as a supplement to analytical monitoring and toxicological testing. Where sources of hazardous compounds are identified, they should be treated or contained on site or transported to sites reserved for the treatment and disposal of hazardous wastes. In a project which is now under consideration, it is proposed that a detailed analytical survey be made of industrial discharges to the sewerage system which serves the Daspoort sewage works in Pretoria, which supplies the feed water to the Stander reclamation plant.

The present situation in South Africa is unfortunately so that the only control which local authorities exercise over the discharge of industrial effluents to their sewers, is that it should not prevent them from complying with the general standards for effluents which had been laid down under the Water Act. The general standards do not include specifications for particular classes or organic compounds.

Planning

Once again considerating the Windhoek example, advance planning played an important role in establishing the reclamation scheme. When a new sewage works had to be constructed, it was with commendable foresight sited close to the existing water purification plant, which enabled the subsequent integration of the reclamation and water purification plants. Likewise, the early segregation of industrial and domestic effluents by the city of Cape Town, and the siting of the new Cape Flats sewage works, will also facilitate eventual reclamation for potable and industrial use.

In a region such as the PWV-complex, however, urban development is under the control of many different local authorities, with the result that the development of effluent treatment facilities and outfall sewers has taken place without co-ordination and the consideration of regional needs. The area produces almost 50% of the gross national product; water demand is high and treated sewage and industrial effluents as well as stormwater runoff are to an increasing extent finding their way to the Barrage in the Vaal River which constitutes the most important source of water supply. It has, in fact, been estimated that at present an average of approximately 35% of the water abstracted from the Barrage consists of purified sewage effluent and industrial wastewater discharges, so that a high degree of indirect reclamation is already taking place.

In order to generate information which could guide planners in the rationalisation of water pollution control and the indirect and direct reuse of wastewaters in the area, the Water Research Commission has initiated an intensive study of all aspects relating to water pollution control and water reuse in the area. The Hydrological Research Unit of the University of the Witwatersrand developed a comprehensive mathematical model for the daily variation of mineral salts in the system. The model can predict, for various time horizons, the time series of TDS in the southern system, including any point within the Barrage storage basin itself. The model is also capable of simulating and determining the effect of further industrial development or of any pollution control or water reuse possibilities which may be proposed (24).

Buildup of Mineral Salts

The buildup of salts in recycling systems beyond acceptable limits can only be avoided by limiting the extent of recycling, by providing some form of bleedoff, or by demineralization.

Bleedoff can be feasible if some beneficial use for mineralized waters can be found in the area involved. It had been estimated, for instance, that an equilibrium TDS level of 500 mg l^{-1} can be maintained if 80% of the available sewage effluent in the Johannesburg metropolitan area is reclaimed and 20% used consumptively, say for irrigation (25). In the case of irrigation, however, there is always the real danger of groundwater pollution and the return of highly mineralized seepage to water courses. The idea of creating artificial lakes for recreational purposes, to which mineralized waters can be discharged, has been put forward for the PWV-complex, but as yet no feasibility studies have been made.

The inclusion of a demineralization stage in a reclamation system will effect a severe cost escalation. With this in mind, the Water Research Commission initiated two projects to evaluate desalination as an AWT process in its own right. The Department of Chemical Engineering, University of Cape Town, has developed an ion exchange process for the simultaneous removal of organic and inorganic compounds from secondary sewage effluent. Following successful laboratory-scale testing, a 5 000 l d^{-1} pilot plant was constructed and extensively tested on water from the Athlone and Milnerton sewage works. This confirmed the technical feasibility of the process for water that has received adequate biological pretreatment. It was shown that prolonged contact with purified sewage effluent does not cause any loss of resin exchange capacity. Resins have been subjected so far to an equivalent of three years of full-scale plant operation without any deterioration in performance capabilities (24). In collaboration with a large chemical concern, a 100 m^3 d^{-1} pilot plant will now be constructed at the Cape Flats sewage works to develop design and operational criteria for the full-scale application of the process.

In the second project, the NIWR is investigating reverse osmosis and ultrafiltration for the desalination and concurrent improvement in organic and microbiological quality of purified sewage effluents. With reverse osmosis, salts are reduced to about 10% of their initial level and COD concentrations of less than 5 mg l^{-1} are consistently achieved. In addition, bacteriological quality, colour and turbidity of the water are significantly improved. Results relating to the life of the membranes have also been encouraging. Problems with membrane fouling and cleaning are still being experienced and are receiving serious attention. Ultrafiltration, although initially promising, did not come up to expectations and work has been suspended pending the development of local membranes with better salt and low molecular mass organic molecule rejection characteristics (24).

Cost Aspects

The technology for upgrading wastewaters to virtually any degree of purity is available, but applications will be determined by the cost of achieving specific quality requirements.

Much of the South African research on water reclamation over the last decade was aimed at the optimization of unit processes and the evaluation of various process configurations, with a view to improving the economics of renovation while at the same time maintaining a high quality product water.

A significant finding by the NIWR, based on experimental runs at the Stander plant, was that savings of between 20% and 30%, depending on the size of plant, could be achieved if activated sludge effluent is used as feedwater instead of biofilter humus tank effluent (26). In treating activated sludge effluent, the quality equalization and ammonia stripping stages could be eliminated from the treatment sequence: lime clarification - quality equilisation - ammonia stripping - recarbonation - sand filtration - breakpoint chlorination - active carbon adsorption - stabilization. Further research has indicated the beneficial effect of ozonation on extending the life of the activated carbon columns with a concomitant reduction in the unit cost of reclamation (27). The economic advantage of carbon regeneration compared to replacement with new carbon was also confirmed. Studies now in progress where lime is replaced by ferric chloride in the primary clarification process, seems to indicate that further savings can be achieved.

The average price charged for water by the large municipalities in South Africa is 20 c m^{-3}. The projected cost of reclaimed water, based on research with the Stander plant, is less than 15 c m^{-3} on a 50 000 m^3 d^{-1} scale, excluding distribution costs.

CONCLUSION

Although the emphasis in this paper has been on the quantitative advantages that can be derived from water reuse technology, it is important to realize that water reuse can be a powerful tool in combating water pollution.

In South Africa, at least, we can no longer afford to depend on dilution to maintain the quality of water in rivers and dams. Indirect reuse through the discharge - dilution - abstraction cycle necessarily degrades the quality of the receiving waters and if these waters are limited and already subject to pollution from many non-point sources, direct reclamation for industrial and even potable use should be encouraged. The direct route will divert at least some effluents and prevent them from reaching and polluting the water environment. The ideal of universal closed cycle operations can probably never be reached, but should be striven after within the framework of technological and economical restrictions.

REFERENCES

1. KRIEL, J.P. Planning and progress in the development of the water resources in South Africa, with special reference to South Western Cape. Congress of the South African Association for the Advancement of Science, Stellenbosch (June 1978).
2. REPUBLIC OF SOUTH AFRICA. Report of the Commission of Enquiry into Water Matters. R.P. 34/1970 (1970).
3. RUDD, R.T. Water pollution control and industrial use of water in South Africa, including reuse of effluents. Group Discussion on Water Reuse, organized by the Iranian Ministry of Power and Energy, Tehran, (1-2 November 1977).
4. HART, OLIVER O. & VAN VUUREN, LUCAS R.J. Water reuse in South Africa, Water Renovation and Reuse (Shuval, H.I., editor). Academic Press Inc., New York (1977).
5. FULS, P.F. & DU PLESSIS, H.M. Effluent disposal in South Africa by means of irrigation. Group Discussion on Water Reuse, organized by the Iranian Ministry of Power and Energy, Tehran, (1-2 November 1977).
6. VAN ROOYEN, G.I.; SMITH, L.S. & STANDER J.v.R. Health requirements of purified sewage effluents for irrigation. Symposium on Recycling of Wastewater for Agricultural and Industrial Uses, Herzliya, Israel (24-26 November 1975).
7. BOLITHO, V. Irrigation farming. Report to City Engineer's Depart=

ment, City of Johannesburg (1970).

8. HYAM, A.F.S. Disposal of factory liquid effluent - an agricultural scheme. IUPAC Symposium on the Chemical Control of the Human Environment, Johannesburg (14-18 July 1969).

9. HENZEN, M.R. & FUNKE, J.W. The appraisal of some significant prob= lems in the reuse of wastewater. J. Inst. Water Poll. Contr. 70, 177-186 (1971).

10. OSBORN, D.W. Factors affecting the use of purified sewage effluents for cooling purposes. Wat. Poll. Control, 456-464, (1970).

11. FLOOK, R.A. Problems associated with the reuse of purified sewage effluents for power station cooling purposes. IAWPR Specialized Conference on the Advanced Treatment and Reclamation of Wastewater, Johannesburg (June, 1977).

12. FUNCKE, J.W. & VAN VUUREN, L.R.J. Water conservation, reuse of effluent and prevention of pollution in the South African industry. Group Discussion on Water Reuse, organized by the Iranian Ministry of Power and Energy, Tehran (1-2 November 1977).

13. GIAMPETRI, A.; FUNKE, J.W. & VOYSEY, J.A. The reclamation of pro= cess water from sewage effluent and the reuse of mill effluents and solids at Mondi Paper Company, Durban, South Africa. IAWPR Specialized Conference on the Advanced Treatment and Reclamation of Wastewater, Johannesburg (June, 1977).

14. LABURN, R.J. & WELLS, R.J. The supply and treatment of water for the Pretoria-Witwatersrand-Sasolburg region. IUPAC Symposium on the Chemical Control of the Human Environment, Johannesburg (14-18 July 1969).

15. VAN EEDEN, W.N. Power station effluent control and the reuse of ash water for cooling water treatment. Water Poll. Contr., 74, (2), 211-215 (1975).

16. STANDER, G.J. & FUNKE, J.W. Conservation of water by reuse in South Africa, Chem. Eng. Prog. Symp. Ser., 63:78 (1967).

17. ALEXANDER, A.T. Control of liquid effluent from African Explosives and Chemical Industries Limited, Modderfontein Factory. IUPAC Symposium on the Chemical Control of the Human Environment, Jo= hannesburg (14-18 July 1969).

18. VAN RENSBURG, J.F.J.; VAN ROSSUM, P.G. & HATTINGH, W.H.J. The oc= currence and fate of micro-pollutants in a water reclaimed for potable reuse. IAWPR Specialized Conference on the Advanced Treatment and Reclamation of Wastewater, Johannesburg (June 1977).

19. VAN RENSBURG, S.J.; HATTINGH, W.H.J.; SIEBERT, M.L. & KRIEK, N.P.J. Biological testing of water reclaimed from purified sewage ef= fluents. IAWPR Specialized Conference on the Advanced Treatment and Reclamation of Wastewaters, Johannesburg (June 1977).

20. NUPEN, E.M. & HATTINGH, W.H.J. Health aspects of reusing wastewater for potable purposes - South African experience. Workshop on Research Needs for the Potable Reuse of Municipal Wastewater, Boulder, Colorado (17-20 March 1975).

21. NATIONAL INSTITUTE FOR WATER RESEARCH. Annual Report 1977/78. CSIR Special Report WAT 56 (1978).

22. WORLD HEALTH ORGANIZATION INTERNATIONAL REFERENCE CENTRE FOR COMMUNITY WATER SUPPLY. Health effects relating to direct and indirect reuse of wastewater for human consumption. Report of an International Working Meeting. Amsterdam (13-16 January 1975).

23. VAN RENSBURG, J.F.J. & HASSETT, A.J. Screening South African drink-ing-waters for organic substances hazardous to health. Water Report (June 1978).

24. WATER RESEARCH COMMISSION. Annual Report 1978.

25. HENZEN, M.R.; VAN VUUREN; L.R.J. & STANDER, G.J. The current status of technological developments in water reclamation. Symposium on Dissolved Solids Loads in the Vaal Barrage Water System. Johannesburg (1970).

26. VAN BLERK, S.H.V.; PRINSLOO, J. & VAN LEEUWEN, J. Comparative reclamation of potable water from biofilter and activated sludge effluents at the Stander water reclamation plant. IAWPR Specialized Conference on the Advanced Treatment and Reclamation of Wastewater. Johannesburg (June 1977).

27. VAN LEEUWEN, J. Personal communication (March 1979).

TABLES

TABLE 1

Daily sewage effluent production and reuse for 33 major cities, towns and industrial complexes in South Africa

Total effluent volume	Irrigation	Power plant cooling	Industrial usage
1 233 500 m^3 100%	198 600 $m^3\ d^{-1}$ 16,1%	107 700 $m^3\ d^{-1}$ 8,7%	87 200 $m^3\ d^{-1}$ 7,1%

TABLE 2

Comparison of daily sewage effluent production and reuse in the PWV-complex and other urban and industrial areas

	Total effluent volume	Irrigation	Power plant cooling	Industrial use
PWV-complex	641 200 $m^3\ d^{-1}$	26,8%	14,4%	8,8%
Other	592 300 $m^3\ d^{-1}$	4,5%	2,6%	5,2%

TABLE 3

Permit holders allowed to dispose of effluents by means of irrigation

Categories of permit holders	Number
Agricultural product processors	
Canners, fruit juice extractors	16
Dairy, sugar and others	7
Tanneries	4
Wineries and distillers	33
Sub total	60
Industries	
Industrial manufacturers	7
Paper mills	5
Textile factories and woolwashers	5
Sub total	17
Mines	
Coal	7
Gold	7
Other	15
Sub total	29
Municipalities (sewage effluent)	134
TOTAL	240

TABLE 4

Reclamation of sewage effluent as cooling water for power stations

	Capacity MW	Reclaimed sewage effluent m^3 d^{-1}
Cape Town (Athlone)	180	10 000
Bloemfontein	102,5	7 500
Johannesburg (Orlando)	300	58 000
Johannesburg (Kelvin)	600	
Pretoria West	180	27 300 (including ash handling)
Pretoria Rooiwal	300	18 200

TABLE 5

Water recycling in four major South African industries

	Water recirculated (m^3 d^{-1})	Intake water (m^3 d^{-1})	Intake %
Steel (ISCOR, Pretoria)	732 000	21 200	2,9
Oil from coal (SASOL)	823 000	63 600	7,8
Chemicals (AECI, Modderfontein)	491 000	17 200	3,5
Thermal Power (Komati)	3 928 000	98 000	2,5

TABLE 6

Water recycling systems at ESCOM power stations

System type	Number of cycles	Stations
Blow-down only	2	Colenso, Hex River, Vaal, Ingange, Umgeni
Blow-down plus H_2SO_4 dosing	3 - 5	Campden, Vierfontein, Klip, Wilge
Pre-clarification plus H_2SO_4 dosing	7 - 10	Grootvlei, Highveld, Taaibos
By-pass sedimentation plus removal of alkalinity	Up to 50	Arnot, Hendrina, Komati, Kriel, Matla* Duvha*

* Under construction

TABLE 7

Effect of water recycling at ESCOM power stations

Year	Water abstracted m^3 d^{-1}	Output TWh	Specific water consumption l/kWh
1965	257 000	23,1	4,1
1975	492 000	65,5	2,7

TABLE 8

Plants featuring in South African research on water reclamation

Plant	Capacity $m^3 d^{-1}$
Windhoek	5 400
Stander experimental plant (Daspoort, Pretoria)	4 500
NIWR pilot plant (Daspoort, Pretoria)	80
Athlone pilot plant (Cape Town)	300
Cape Flats experimental plant	4 500

THE RECLAMATION OF INDUSTRIAL/DOMESTIC WASTEWATERS

Lucas R.J. van Vuuren
Assistant Director
National Institute for Water Research, Council for Scientific and
Industrial Research, P.O. Box 395, Pretoria 0001, Republic of
South Africa

Mark P. Taljard
Works Chemist
City Engineer's Department, City of Cape Town, P.O. Box 1694,
Cape Town 8000, Republic of South Africa

INTRODUCTION

Wastewater reuse is gaining recognition as an option for augmenting local water supplies. During recent years reclamation technology and pollution control measures have received increasing attention in various parts of the world. In South Africa several pilot and production-scale advanced treatment facilities have been yielding results since the early sixties and a great deal of design and operational experience has been gained in treating wastewaters of various types and qualities.

A review of these activities reveals that the interrelationships between wastewater quality, type of treatment facility, quality of secondary effluent and advanced treatment process requirements were of major importance. In the case of trickling filter plants, the secondary effluents showed greater quality fluctuations than effluents from activated sludge plants. The major problems encountered were associated with factors such as climatic conditions, loading rates and also, to a large extent, the presence of industrial effluents. These effluents often gave rise to poor biodegradability which resulted in high ammonia and organic carbon residuals with wide diurnal and seasonal fluctuations. For reclamation purposes they posed considerable problems mainly in terms of disinfection control and efficient organic carbon removal.

More recently the National Institute for Water Research (NIWR) extended its research on reclamation technology to cover integrated physical-chemical and biological processes to provide for the integration of wastewater treatment and reclamation into one engineering discipline.

This paper deals essentially with pilot plant studies at Pretoria using combined physical-chemical-biological methods for the renovation of wastewater mainly of domestic origin. Results obtained from a second pilot-scale study (Athlone plant) with wastewater containing a higher proportion of industrial effluents are also briefly discussed.

QUALITY AND PROCESS CONSIDERATIONS

For unrestricted reuse of wastewater, as in the case of the City of Windhoek, it was regarded as essential that the wastewater should be predominantly domestic in origin. Nitrogenous compounds from the local abattoir and meat-packing industries were duly separated before the secondary effluent, which was also passed through a series of algae maturation ponds, was considered amenable to advanced treatment(1). In the long run, however, the trickling filters and algae maturation ponds proved to be unreliable for securing consistently low ammonia residuals which, in turn, were essential for effective disinfection. High lime treatment with partial ammonia stripping was subsequently implemented to ensure better utilisation of the Windhoek reclamation facilities(2).

A similar water reclamation plant at Daspoort served as a research and demonstration facility for reclamation of effluents derived initially from trickling filters and subsequently from activated sludge. In the case of the trickling filters, a balancing and quality equalisation basin was found highly advantageous for improved ammonia removal and fail safe disinfection control(3). The use of activated sludge effluents as opposed to trickling filter effluents was found to be much superior as an intake source for reclamation purposes in terms of smaller quality fluctuations, reduced ammonia and residual COD.

INTEGRATED PHYSICAL-CHEMICAL-BIOLOGICAL TREATMENT

The NIWR has developed a 60 m^3/day pilot plant for the treatment of screened raw sewage with a relatively low industrial effluent component. This facility was flexibly designed to enable various process configurations and operational modes to be compared. Extensive studies with high lime treatment and dissolved air flotation followed by biological treatment (LFB process) have been reported(4,5). More recent studies have included the use of ferric chloride as primary coagulant and partial denitrification with a view to cutting down on the chemical and maintenance costs experienced in high lime treatment.

Plant Description (Pretoria)

A flow diagram of the modified LFB plant is shown in Figure 1 and the individual process units are briefly described below. The process stages may be classified into biological, chemical and polishing stages. For the purpose of this paper, the performance of the biological and chemical units is discussed in greater detail because of their overriding importance in the process as a whole.

Denitrification Reactor. Raw screened sewage enters a reactor vessel of 16 m^3 capacity at a rate of 40 ℓ/min. Recycled streams from several succeeding stages are returned to the reactor. These include fully nitrified effluent together with chemical organic sludge and waste activated sludge. The recycled sludges serve as a source of carbon and for seeding of organisms for the purpose of partial biological denitrification. The recycle rate of nitrified effluent affects the extent to which denitrification is achieved. A ratio of 1:1 effectively reduces final nitrate to about 12 mg/ℓ as N. Limited runs with a recycle ratio of 1.5:1 confirmed that the nitrate could be controlled at less than 8 mg/ℓ.

Chemical Clarifier. Ferric chloride is dosed to the blended streams of the reactor outflow and lime added to control pH. Sludge is drawn continuously from the chemical clarifier (24 ℓ/min) and split into three equal streams of which one is returned to the reactor unit, one to the chemical flocculation tank and one wasted to the thickener prior to anaerobic digestion (Figure 2). The waste sludge serves as a reserve carbon source during periods of weak sewage strength. Night time flows in particular are low in organic content and are best treated by not wasting the chemical organic sludge.

Ferric chloride dosages (as $FeCl_3$) based on the inflow rate of raw sewage can be cut from an initial 160 mg/ℓ to 75 mg/ℓ as a result of continuous recyling of chemical sludge. Low turbidity (<2.0 JTU) in the effluent from the chemical clarifier is the criterion used which also coincides with efficient phosphate removal.

Nitrification Pond. An aeration pond and secondary clarifier serves as an activated sludge process stage. Addition of a further dosage of ferric chloride (10 mg/ℓ) enhances the settleability of the activated sludge and secures a low effluent turbidity for further polishing.

The results reported in this paper are representative of a sludge age of 15 days with an MLSS and VSS corresponding to about 1 100 and 600 mg/ℓ respectively (Figure 3). These results confirm that nitrification was highly efficient with low residual COD and ammonia levels.

The aeration pond was designed, for a nominal retention of 7½ hours, based on a raw intake flow of 130 m^3/d during earlier runs with the high lime treatment. This flow had to be reduced to 60 m^3/d to allow for recycle and hydraulic limitations of the clarifiers whereby the nominal retention in the aeration basin was inadvertently increased to about 16 hours.

Polishing Process Units. The effluent from the activated sludge clarifier with a turbidity of less than 2 JTU, is prechlorinated and passed through a roughing filter, followed by a dual media filter (turbidity <0.6). The filtered effluent is finally polished by means of a single active carbon packed column and free residual chlorination is applied for final disinfection. In view of the low chlorine demand and low organic carbon content the formation of halogenated hydrocarbons is expected to be minimised.

Hydraulic details of the primary process units excluding the polishing units are summarised in Table 1.

Operational Results (Pretoria)

Chemical Quality. Average weekly results corresponding to a period of sixteen weeks of continuous operation are presented in Table 2 and the improvement in quality through the chemical-biological stages is illustrated in Figure 4.

The effluent from the activated sludge stage was allowed through the polishing units only during special runs for the purpose of microbiological monitoring and final quality assessment. A typical quality of the product water achieved during these runs is shown in Table 3.

The effluent from the activated sludge clarifier was found to be consistently of a very high average quality in terms of COD (<30 mg/ℓ), NH_3 (0.2 mg/ℓ as N) and soluble phosphate (<0.2 mg/ℓ as P). This quality proved to be excellent for further polishing in that the chlorine demand for complete disinfection was extremely low (<3 mg/ℓ). This finding is considered to be of great significance in that multiple safety barriers can be created by successive chlorination points without extravagant costs. In this particular study the two chlorination points (total of 9 mg/ℓ) proved most satisfactory for disinfection control purposes. The low COD prior to active carbon polishing is also indicative of lower active carbon requirements as compared with effluents from trickling filter plants.

The low phosphate of less than 0.2 mg/ℓ as P which was consistently achieved is also of great significance from a pollution control viewpoint.

Microbiological Quality. The effluent at various stages of processing was monitored weekly for eight consecutive weeks (Table 4).

Owing to the recycling of biomass to the chemical clarifier, this unit process did not achieve any dramatic removal of microorganisms. A marked improvement was, however, effected in the secondary biological clarifier where up to two logs in total plate count, coliforms and coliphage were recorded. By prechlorination coliphage was completely absent during all tests and TPC and coliforms were reduced to extremely low values.

After active carbon polishing a two log increase in TPC was recorded, evidently as a result of biological growth on the media, but this was effectively reduced to zero order levels by final chlorination.

Denitrification. A balance of NO_3 plus NO_2 relating to the reactor influent and the chemical clarifier effluent showed virtually complete denitrification (>90%) with an average residual nitrate of 0.6 mg/ℓ as N. The reduction of total dissolved nitrogen was 58% of which 47.5% was associated with the denitrification/chemical stages and 10.5% with the activated sludge process stage. The polishing stages did not contribute significantly to overall nitrogen reduction.

The rate of denitrification was calculated at 1.1 mg N per gram VSS per hour. The residual nitrate out of the reactor was however found to be critically dependent on the VSS confirming a value in excess of 1000 mg/ℓ for efficient denitrification (Figure 5). The reactor unit has not as yet been optimised in terms of minimum retention and flow regime. A plug flow arrangement is expected to improve its performance further.

The ratio of soluble COD to NO_3-N reduction in the reactor unit was of the order 4.7. An overall reduction of soluble COD of about 77% was achieved up to the activated sludge stage, more than 60% of which was due to the denitrification/chemical stages. On the basis of a total COD of 350 mg/ℓ entering the plant the overall reduction by chemical and biological treatment exceeded 90%.

Nitrification was readily accomplished in the aeration pond with a biomass corresponding to an MLSS and VSS of about 1 000 and 600 mg/ℓ respectively. The activated sludge showed excellent settling properties on account of a further dosage of ferric chloride (10 mg/ℓ) which evidently also ensured phosphate residuals of less than 0.2 mg/ℓ as P.

Chemical-Biological Sludge. The combined chemical-biological and raw organic sludges drawn from the chemical clarifier (see Figure 2) served the following important functions:

1. A source of organic carbon and seeding of microorganisms for denitrification.
2. Improved setteability and reduced ferric chloride dosage, apparently as a result of sludge recycle.

The settleability of the sludge in the chemical clarifier was also significantly improved by ferric chloride dosage (see Figure 6) and upflow velocities exceeding 2 m/hr could readily be achieved in both clarifiers.

Complementary laboratory-scale studies were conducted in order to assess the stability of phosphate in the sludge wasted from the chemical clarifier. Samples of thickened sludge were stored under anaerobic conditions in the laboratory and the release of P, NH_3 and COD monitored with time. After 5 days no release of P could be detected whereas an increase of both NH_3 and dissolved COD confirmed anaerobic biodegradation.

In subsequent tests sludge samples representative of varying ferric chloride dosages were stored for periods of up to two months. Phosphate release into the supernatant was minimal (<1 mg/ℓ) and could be roughly correlated with the iron content of the sludges. The stability of the phosphate bonded sludge is most encouraging in relation to phosphate release in the case of pure biological phosphate bearing sludges. These studies are currently being extended to detailed pilot-scale evaluation which will, amongst others, incorporate criteria for thickening and sludge stabilisation.

ATHLONE PILOT PLANT

Based on the original development of the LFB process, a second pilot plant (300 m^3/d) was constructed for the reclamation of secondary effluent derived from the Athlone sewage works near Cape Town. This particular sewage works receives a stronger sewage which includes wastewaters of industrial origin. Domestic effluents from this area are partially diverted to a separate sewage works for the purpose of eventual unrestricted reuse.

The Athlone works incorporates the use of trickling filters which produce a final effluent of poor quality as a result of overloading. Nitrification and carbonaceous oxidation are accordingly low as reflected by residual COD values in the final effluent of 220 mg/ℓ, ammonia of 36 mg/ℓ as N and nitrate below 1 mg/ℓ as N on the average. The AWT pilot plant was designed to investigate the potential reuse of this quality effluent for selected industries such as textiles.

A flow diagram of the Athlone pilot plant is presented in Figure 7. High lime treatment and partial ammonia stripping are provided as pretreatment stages. This is followed by a surface aerated activated sludge process with ferric chloride dosage to improve secondary clarification. The polishing units comprise pre-chlorination, gravity filtration, active carbon adsorption and free residual chlorination. Hydraulic details of the individual units are listed in Table 5.

Chemical and Microbiological Results

The Athlone AWT plant was commissioned in 1976 and has been in intermittent operation for periods of up to 3 months. A great deal of effort was devoted to the optimisation and refinement of individual process units particularly high lime dosage control, ammonia stripping and gravity filtration.

Representative data of chemical quality improvement through the successive stages are listed in Table 6 and microbiological results in Table 7.

As in the case of the Pretoria plant, extremely low ammonia levels could be maintained with efficient disinfection control.

RUNNING COSTS

The cost of chemicals based on bulk supply prices (1979) and the electrical power costs for aeration, pumping and ammonia stripping for both pilot plants are compared in Table 8 (1 SA cent equals 1.15 US cent).

It should be noted that the total cost for chemicals and electricity for the Athlone plant is about double that for the Pretoria plant. This is essentially due to high lime, carbon dioxide and active carbon requirements and also the need for ammonia stripping in the case of the Athlone plant.

DISCUSSION

Operation of the two separate AWT pilot plants based on integrated physical-chemical-biological principles have confirmed several distinct advantages both from a pollution control and water reclamation point of view:

> Nitrate reduction could be achieved by partial ammonia stripping in the case of the Athlone plant and partial denitrification without addition of an external energy source in the case of the Pretoria plant. In both cases phosphate could be removed to acceptably low residuals.
>
> Chemical pretreatment consistently ensured efficient nitrification and carbonaceous oxidation. This had a beneficial effect on disinfection control and active carbon utilisation. Aeration energy and retention time in the aeration basin have not been optimised during these studies but indications are that much lower requirements compared with conventional activated sludge are achievable. Both plants performed satisfactorily with a nominal retention of 7½ hours and sludge age of 15 to 25 days. In the case of the Athlone plant a 25 day sludge age resulted in an MLSS concentration of between 500 and 1 500 mg/ℓ. Values of less than 100 mg/ℓ MLSS were occasionally recorded without loss of nitrification efficiency.
>
> For the reclamation of sewage contaminated with industrial effluents, the integrated approach offers distinct advantages in that the activated sludge process stage is protected by chemical pretreatment(6). The use of ferric chloride as primary coagulant ensures less operational control problems than high lime treatment and from a cost point of view was found to be more favourable. An increase in total dissolved solids with associated hardness should however be weighed against these advantages.
>
> Conversion of the high lime process to a ferric chloride coagulation system at the Pretoria plant has been highly successful and is also being considered for Athlone.

CONCLUSIONS

The two pilot-scale studies reported in this paper have confirmed the suitability of integrated chemical-biological systems for the reclamation of wastewaters of varying quality. Effluents containing an appreciable proportion of industrial contaminants could be rendered more amenable to biological oxidation by chemical pretreatment. Both lime treatment and ferric chloride have been used successfully but the latter coagulant offers the advantage of partial biological denitrification, better operational control and lower treatment costs.

A detailed evaluation of quality parameters relating to health aspects and optimum design criteria form part of the current research program. This includes mathematical modelling of the main process parameters.

ACKNOWLEDGMENTS

The assistance of the Cape Town City Engineer's Department is greatly appreciated. A special word of thanks is due to Dr S.G. Wiechers of the NIWR staff for his enthusiastic participation. Thanks are also due to the Director of the NIWR for permission to publish this paper and to the Water Research Commission for financial support of the Athlone plant.

REFERENCES

1. VAN VUUREN, LUCAS R.J.; HENZEN, M.R.; STANDER, G.J. & CLAYTON, A.J. The full-scale reclamation of purified sewage effluent for the augmentation of the domestic supplies of the City of Windhoek. Paper presented at the 5th International Water Pollution Research Conference, San Francisco, 26th July - 5th August 1970, pp. 1-32/1-1-32/9.

2. VAN VUUREN, LUCAS R.J.; CLAYTON, A.J. & VAN DER POST, D.C. Current status of water reclamation at Windhoek. Paper presented at 51st Water Pollution Control Federation Annual Conference, Anaheim, 1-6 Oct. 1978.

3. VAN VLIET, BEN M.; WIECHERS, H.N.S. & HART, O.O. The efficacy of an equalization pond in a water reclamation system. Paper presented at the 8th International Water Pollution Research Conference, Sydney, Australia, 17-22 Oct., 1976. Progress in Water Technology, 9:2:443 (1977).

4. VAN VUUREN, LUCAS R.J.; ROSS, W.R. & PRINSLOO, J. The integration of wastewater treatment with water reclamation. Paper presented at the 8th International Conference of the IAWPR, Sydney, Australia, 17-22 Oct., 1976. Progress in Water Technology, 9:2:455 (1977).

5. VAN VUUREN, LUCAS R.J. & WIECHERS, S.G. Nitrification and denitrification of lime-treated wastewaters. Paper presented at the International Conference on Advanced Treatment and Reclamation of Wastewater, Johannesburg, 13-17 June, 1977. Progress in Water Technology, 10:1/2:185 (1978).

6. CILLIE, GABRIEL G.; DAVIES, T.R.; HART, O.O. & HASSETT, A.J. The effect of toxic loads on effluent purification systems. Paper presented at monthly meeting of the Institute of Water Pollution Control (S.A. Branch), 29 April 1976. Water S.A., 3:2:83 (1977).

TABLE 1

Chemical Biological Process Units : Hydraulic Loadings

Process Unit	Volume (m^3)	Nominal Flow (ℓ/min)	Nominal Reten-tion (hr)	Flow with Recycle (ℓ/m)	Reten-tion with Recycle (hr)	Over-flow Rate (m/hr)
Denitrification Reactor	16	40	6.7	88	3.0	0.9
Chemical Clarifier	2.5	40	1.0	72	0.6	2.2
Nitrification Pond	40	40	16.7	95	7.0	0.1
Biological Clarifier	2.5	40	1.0	78	0.5	2.3

TABLE 3

Quality of Raw and Final Product Water

Quality Parameter		Raw Screened Sewage	Final
Turbidity (JTU)		8	<0.6
Colour (Hazen)		120	8
pH		7.3	7.8
Electrical Conductivity	(mS/m)	69	67
Na	(mg/ℓ)	45	40
K	(mg/ℓ)	12	10
Ca	(mg/ℓ)	38	55
Mg	(mg/ℓ)	20	22
TKN	(mg/ℓ)	28	<0.5
NH_3-N	(mg/ℓ)	18	<0.2
$NO_3 + NO_2$	(mg/ℓ)	<1	11.0
Si	(mg/ℓ)	8	6
SO_4	(mg/ℓ)	70	35
Total-P	(mg/ℓ)	6.0	<0.2
Ortho-P	(mg/ℓ)	4.1	<0.2
Cl	(mg/ℓ)	50	100
Total Alkalinity (as $CaCO_3$	mg/ℓ)	220	102
COD	(mg/ℓ)	350	<10
LAS	(µg/ℓ)	3 600	620
Al	(µg/ℓ)	120	<100
Bo	(µg/ℓ)	300	100
Phenol	(µg/ℓ)	<10	<10
F	(µg/ℓ)	<100	<100
Fe	(µg/ℓ)	470	<100

TABLE 2

Chemical Analysis : Weekly Averages on Filtered Samples Composited Daily (mg/ℓ)

	Raw Screened Sewage						Chemical Clarifier						Biological Clarifier					
Weeks	NO_x-N	NH_3-N	TKN	Tot-N	COD	PO_4-P	NO_x-N	NH_3-N	TKN	Tot-N	COD	PO_4-P	NO_x-N	NH_3	TKN	Tot-N	COD	PO_4-P
1	0.9	19.1	-	-	105	3.4	0.5	9.2	-	-	46	0.3	12.1	0.9	-	13.0	36	0.5
2	1.1	20.9	28.7	29.8	-	4.7	1.3	10.8	13.8	15.1	64	0.8	12.0	0.9	-	12.9	36	0.6
3	1.2	18.7	27.0	28.2	131	5.0	0.6	10.2	10.8	11.4	44	0.1	11.5	0.1	-	11.6	33	0.1
4	1.0	17.8	27.1	28.1	99	5.2	0.6	9.8	12.1	12.7	39	0.0	12.2	0.1	-	12.3	30	0.0
5	1.0	18.2	27.5	28.5	136	5.8	0.2	9.3	12.9	13.1	40	0.2	13.3	0.0	-	13.3	23	0.1
6	1.2	16.9	28.6	29.8	105	4.1	1.6	9.0	13.8	15.4	39	0.0	12.5	0.2	-	12.7	27	0.0
7	1.2	18.2	30.9	32.1	123	3.5	0.6	10.0	13.6	14.2	39	0.0	11.1	0.0	-	11.1	24	0.0
8	1.2	18.0	27.6	28.8	108	3.4	0.6	8.8	11.6	12.2	38	0.2	11.4	0.2	-	11.6	23	0.0
9	0.8	16.9	24.8	25.6	101	3.3	0.5	8.9	11.7	12.2	34	0.2	11.1	0.2	-	11.3	21	0.0
10	0.6	18.4	26.0	26.6	110	4.1	0.6	8.9	11.8	12.4	33	0.0	11.3	0.2	-	11.5	19	0.0
11	0.4	18.2	28.2	28.6	129	3.7	0.4	9.8	13.1	13.5	45	0.5	11.5	0.2	-	11.7	20	0.04
12	0.8	16.8	30.0	30.8	101	3.1	0.6	10.0	14.8	15.4	34	0.2	11.6	0.8	-	11.8	19	0.3
14	1.1	14.9	27.3	28.4	107	4.2	0.6	9.5	14.6	15.2	43	1.5	12.7	0.2	-	12.9	23	0.1
15	1.2	15.4	25.3	26.5	103	4.4	0.6	8.0	12.5	13.1	38	0.8	11.6	0.0	-	11.6	27	<0.2
16	1.2	16.3	25.9	27.1	117	4.3	0.6	7.5	12.3	12.9	37	0.4	11.1	0.0	-	11.1	27	<0.2
Average	1.0	17.5	27.5	28.5	112	4.1	0.6	9.6	12.9	13.6	41	0.3	11.8	0.2		12.0	26	0.13

TABLE 4

Microbiological Results

Quality Ranges of Weekly Monitoring

Test		Raw	Chemical Clarifier	Biological Clarifier	Prechlo-rination	Filtered	Activated carbon	Final
Total plate count per mℓ		1.6 - 380(x10^5)	2.9 - 140(x10^5)	3.9 - 63(x10^4)	8 - 83	1 - 65	12 - 77(x10^2)	0 - 17
Total coliforms per 100 mℓ		11 x 10^5 - ∞	1.6 - 330(x10^5)	6.6 - 150(x10^3)	0 - 68	0	0 - 1	0
Coliphage per mℓ	118	56 - ∞	103 - 135	7 - 59	0	0	0	0
	K2	81 - 727	19 - 126	4 - 26	0			

TABLE 5

Athlone Pilot Plant - Hydraulic Loadings

Flows:	Raw Water	300 m^3/d	3.47 ℓ/s
	Primary Sludge Recycle	13 m^3/d	0.15 ℓ/s
	Primary Sludge Wastage	8.5 m^3/d	0.10 ℓ/s
	Secondary Sludge Recycle	41.5 m^3/d	0.48 ℓ/s
	Secondary Sludge Wastage	4 m^3/d	0.05 ℓ/s

Unit	Volume (m^3)	Area (m^2)	Loading (m/hr)	Retention Time (Nominal) (hr)	Retention Time (Actual) (hr)
Lime Reactor	3.2	-	-	0.26	0.25
Chemical Clarifier	40	14	0.89	3.2	3.1
Aeration Basin	92	-	-	7.4	6.5
Biological Clarifier	40	14	0.89	3.2	2.8
Sand Filter	4	3.1	4.0	0.32	-
Chlorine Contact Tank	15	-	-	1.2	1.2
Carbon Column 1	4	1.2	10.4	0.32	± 0.1
Carbon Column 2	4	1.2	10.4	0.32	± 0.1
Final Effluent Tank	15	-	-	1.2	1.2

TABLE 7

Athlone Pilot Plant - Bacteriological Data

	Raw Water	Chlorine Contact Tank
Total coliforms @ 37 °C MPN per 100 mℓ	$2.4 \times 10^6 - 2.1 \times 10^9$	Zero
Presumptive *E. coli* @ 44.5 °C MPN per 100 mℓ	$1.5 \times 10^5 - 4.6 \times 10^8$	Zero
Clostridium perfringens	$4 \times 10^2 - 1.1 \times 10^5$	Zero
Faecal streptococci	$1.4 \times 10^4 - 1.4 \times 10^5$	Zero
Pseudomonas aeruginosa	$1.1 \times 10^2 - 1.1 \times 10^3$	Zero
Coliphage	$1.5 \times 10^3 - 3.2 \times 10^5$	Zero

TABLE 6

Athlone Reclamation Plant - Average Analyses

		Final Effluent	Primary Clarifier Effluent	Ammonia Stripper Effluent	Secondary Clarifier Effluent	Sand Filter Effluent	Final Water
Ammonia as N	mg/ℓ	36.2	33.3	17.3	0.2	0.2	0.2
Organic Nitrogen	mg/ℓ	9.5	4.3	4.2	3.0	2.0	0.6
Nitrite as N	mg/ℓ	0.2	0.6	2.3	0.2	-	0.03
Nitrate as N	mg/ℓ	0.8	1.2	2.6	22.6	-	23.1
COD	mg/ℓ	220	91		65	52	22
Dissolved Solids	mg/ℓ	785	-	-	-	-	800
pH		-	-	-	-	7.2	6.8
Chloride as Cl	mg/ℓ	-	-	-	-	-	233
Total phosphorus as P	mg/ℓ	10.2	0.8	-	0.85	0.7	0.8
Calcium as Ca	mg/ℓ	37	55	-	57	67	63
Magnesium as Mg	mg/ℓ	15	4	-	4	4	4
Iron as Fe	mg/ℓ	1.1	1.8	-	1.3	0.2	0.3
Manganese as Mn	mg/ℓ	0.14	0.04	-	0.04	0.03	0.02
Sulphate as SO_4	mg/ℓ	99	-	-	-	-	84
Colour (Hazen)		205	68	-	58	50	9
Turbidity, JTU		-	-	-	-	1.2	1.0

TABLE 8

Running Cost for Integrated Systems
(Electrical and Chemicals only)

	Pretoria Plant 60 m^3/d			Athlone Plant 300 m^3/d		
Item	Price Delivered c/kg	Dosage mg/ℓ	Cents/m^3	Price Delivered c/kg	Dosage mg/ℓ	Cents/m^3
Ferric chloride (as 100% $FeCl_3$)	12.2	85	1.04	36.8	37	1.37
Lime [as $Ca(OH)_2$]	4.5	70	0.32	5.0	482	2.41
Polyelectrolyte	265	0.75	0.20	275	1.7	0.45
Chlorine	44	9	0.40	53	11.2	0.59
Carbon dioxide	-	-	-	85	6.4	0.55
Activated carbon	-	-	4.00*			6.00+
Total Chemicals			5.96			11.37
Power	2.1 c/kWhr		0.72**	2.76 c/kWhr		2.40++
Total Chemical and Electrical			6.68			13.77

* Projected from full-scale reclamation plant
** Projected from full-scale activated sludge plant
+ Projected from full-scale reclamation plant and higher intractable organic load
++ Projected from full-scale activated sludge plant and including ammonia stripping

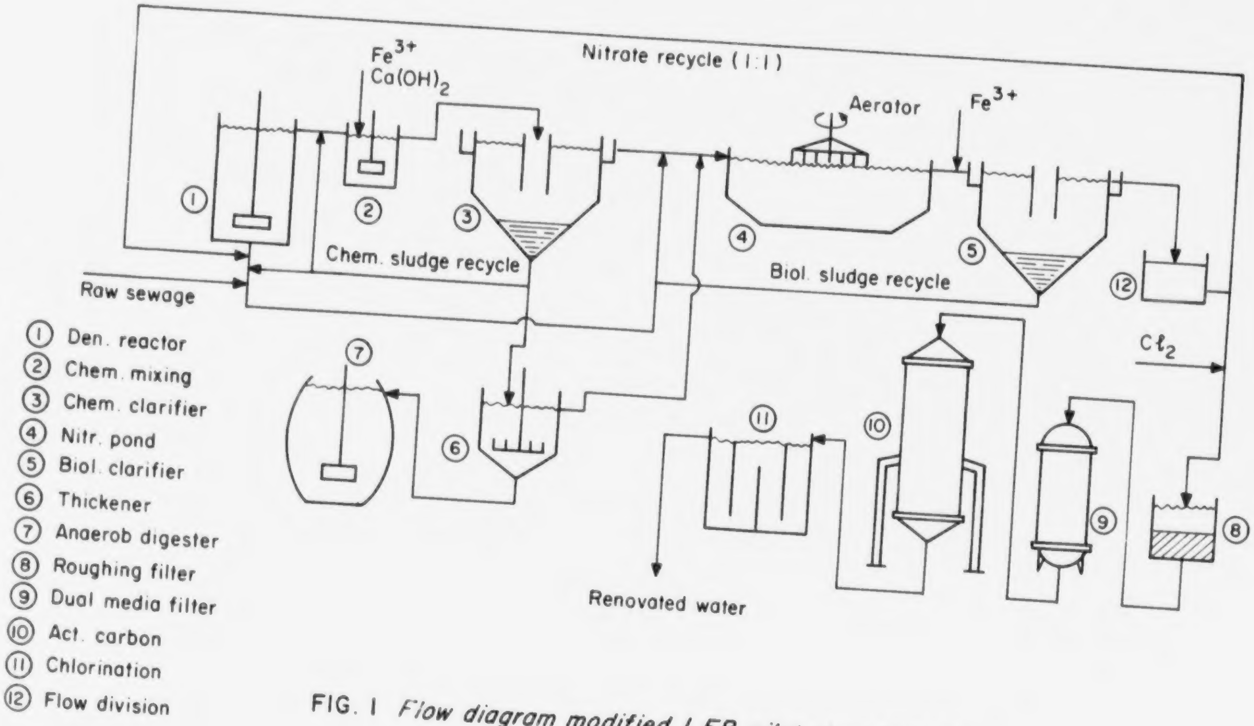


FIG. 1 *Flow diagram modified LFB pilot plant (60 kℓ/d)*

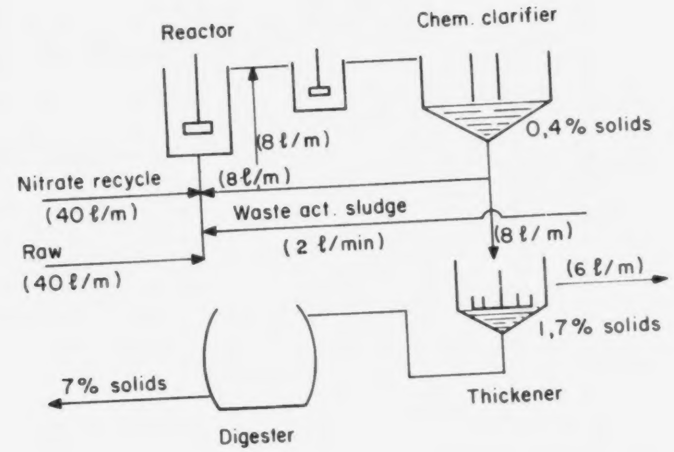


FIG. 2 *Flow rates and concentration of sludges.*

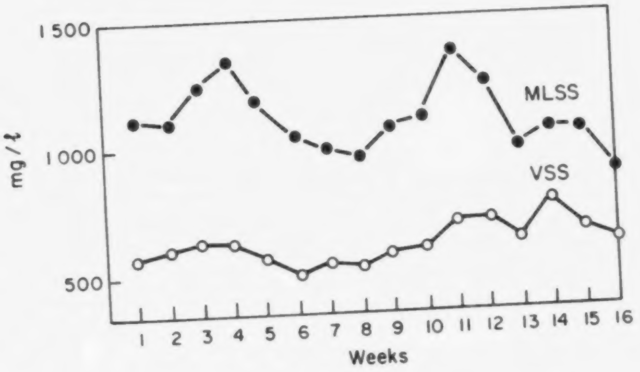


FIG. 3 *Activated sludge properties.*

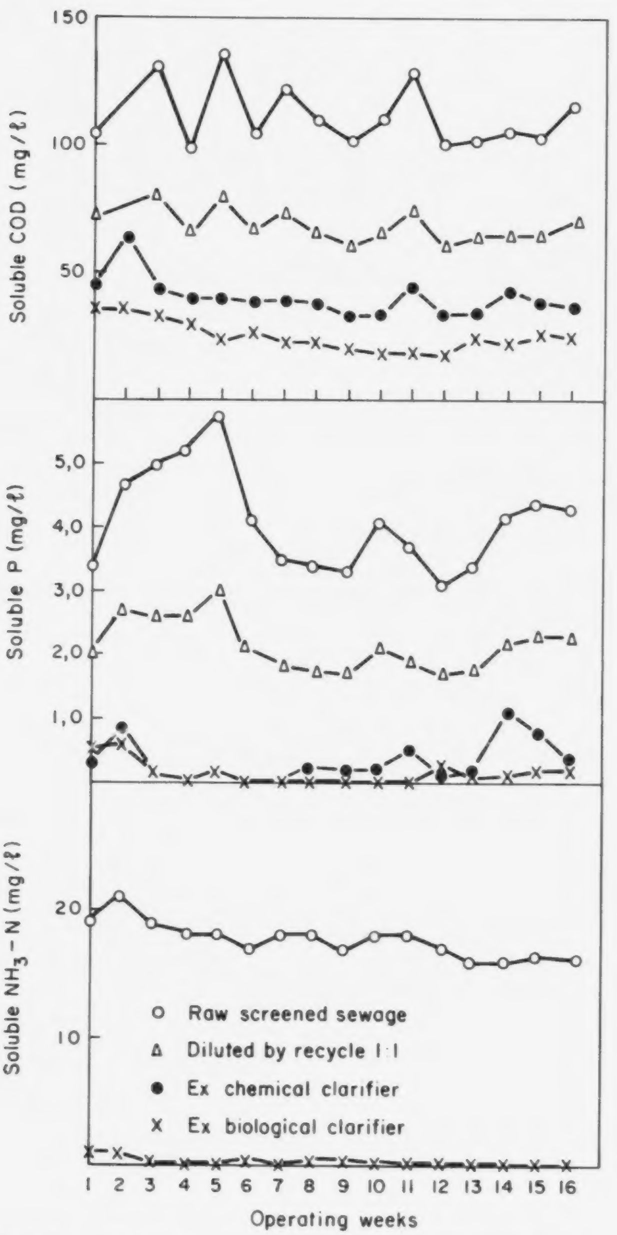


FIG. 4 *Quality improvement through chemical and biological stages.*

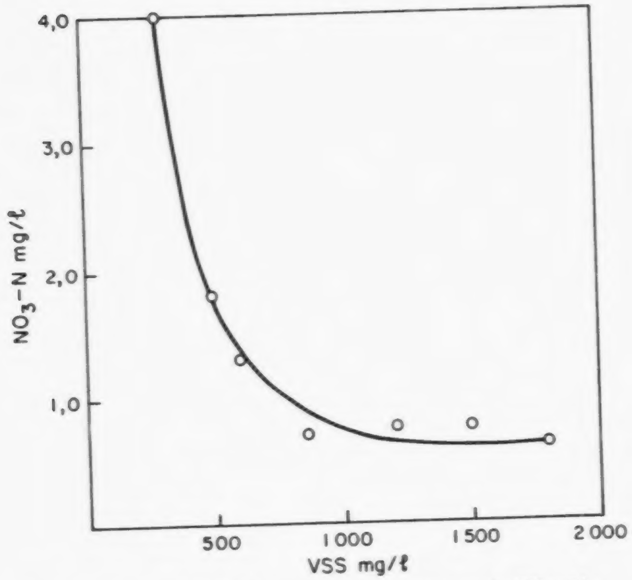


FIG. 5 *Effect of biomass conc. on denitrification.*

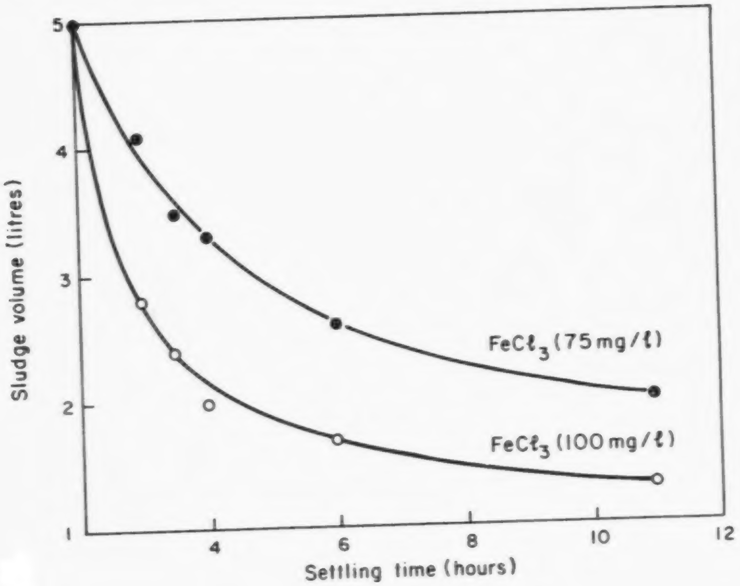


FIG. 6 *Sludge settling curves.*

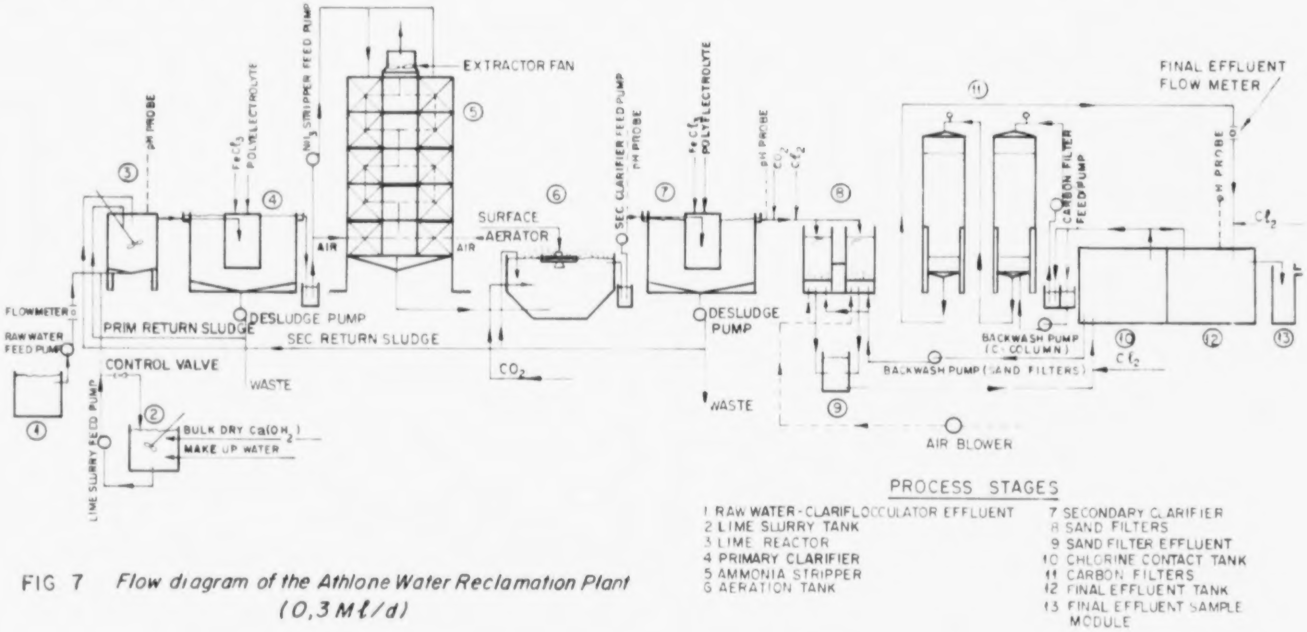


FIG 7 *Flow diagram of the Athlone Water Reclamation Plant (0,3 Mℓ/d)*

RE-USE OF WATER IN AUSTRALIA

M. A. SMITH
PROJECTS ENGINEER
MINISTRY OF WATER RESOURCES AND WATER SUPPLY
VICTORIA AUSTRALIA

INTRODUCTION

The continent of Australia covers an area of 7,682,000 km^2 and experiences a wide range of climates - from tropical rainforest in Northern Queensland to arid desert in Central Australia.

It is well known that Australia is the driest continent on earth. It is not as well known that Australia's rivers generally exhibit higher variability in flow than European and American rivers. For example the co-efficient of variation of Australian rivers ranges from 0.1 to 1.5 while American rivers tend to have co-efficients of variation in the 0.2 to 0.5 range.

Clearly then the development of the nation's water resources presents special difficulties. So far it has been possible to meet water demands by the construction of large storages and, in many cases, by transporting water over long distances.

Australia's population is currently only 14 million and it is obvious that, as the population increases, the provision of adequate water supplies to meet the competing demands will become more difficult. In some areas the situation of full practical utilization of water resources is already close at hand.

Given the situation described above it is hard to understand why the use of reclaimed water is not widespread. However, there are some notable examples of re-use of water and interest in this topic is increasing daily.

The purpose of this paper is to review the present situation with respect to the use of reclaimed water in Australia and to discuss some of the research that is being carried out.

PRESENT DEVELOPMENT

For convenience the use of reclaimed water will be considered under the following categories:

. Irrigation - pasture
    - silviculture
    - horticulture
    - viticulture
. Stream Flow Maintenance
. Landscape Watering
. Groundwater Recharge
. Industrial Use
. Municipal Use
. Domestic Use.

Irrigation

(a) Pasture. The "Werribee Farm" near Melbourne is probably one of the world's best examples of the benefits to be derived from the use of reclaimed water. The farm commenced operations in 1897 and until recently treated ninety percent of Melbourne's sewage.

The farm currently covers an area of some 10,800 ha. and treats an average of 200,000 ML (200 million m^3) of wastewater per year. Of this approximately 20 percent (40,000 ML/a) is used for land irrigation during the summer period, the remainder being treated in lagoons or by grass filtration during the winter.

Approximately 80 percent of the sewage used for irrigation is applied to the land without prior treatment. Sewage is applied at a rate of 100 mm every 18 days.

While the prime purpose of the farm is the efficient and cheap treatment of sewage it has developed into a classic example of the benefits of using reclaimed water. Sheep were first introduced onto the farm in 1900 and cattle some 10 years later. At present the farm carries some 20,000 beef cattle and 50,000 sheep, making it one of the largest pastoral activities in the State. Total income from the sale of stock is currently approximately A$0.5 million per year.

Investigations are currently being carried out with a view to improving the efficiency of treatment at the farm by the use of different crops.

Apart from the Werribee Farm, some 42 local Sewage Authorities scattered throughout Victoria, dispose of treated effluent by land irrigation on a total of 200 ha. of land. In all cases land disposal has been adopted because of its efficiency, cheapness and environmental acceptance rather than the development of a valuable resource. However, in nearly all cases the pasture is used for grazing sheep and, in fourteen cases cattle, the income from sale of stock being used to defray the treatment costs.

In Victoria severe restrictions are placed on the movement of cattle that have been grazed on pasture irrigated with sewage effluent. These restrictions are imposed because of a fear of the spread of beef measles (C.bovis). However, monitoring of carcases has shown that the incidence of C.bovis in Werribee Farm cattle is no greater than the State average.

The Copranapra Pastoral Company irrigates some 300 ha. of fodder crops on the plains north of Adelaide, South Australia. Some 900 Ml. of secondary sewage effluent is used each year. Fodder is used as feed for dairy cattle; milk being sold on the Adelaide market.

(b) Silviculture. This appears to be an ideal way of using sewage effluent due to the minimal health risk involved.

There are no examples of trees being irrigated with reclaimed water in Australia on a commercial basis. However, the number of investigations being carried out in widely separated areas shows the growing interest in this form of re-use.

A stand of tropical hardwoods has been grown under irrigation with effluent from the Darwin sewage treatment lagoons. The plots were originally irrigated for one hour per day and later four hours per day. Sixteen months after planting it was reported that the growth of many species of trees was considerably enhanced by the use of sewage effluent.

Unfortunately the trees were destroyed in the cyclone that hit Darwin, Northern Territory, on Christmas Day, 1974.

A trial is also being carried out near Perth, Western Australia, but unfortunately no progress reports have been released.

Major production trials are currently being carried out at three locations in Victoria. At Horsham and Mildura 4 ha. of various species of Eucalypts were planted in late 1977. Some 3,000 trees have been planted at each site at 1.5 m x 3.0 m spacing. The trees are flood irrigated at intervals so that field capacity of the soil is maintained.

Results to date are very encouraging as indicated by measurements of trees age 12 months at Mildura:-

Trees Age 12 Months

Species	Survival %	Mean Ht. cm	Mean dom.[a] ht. cm	Mean Diameter[b] cm
Eucalyptus globulus	85	248	324	4.9
E. botryoides	76	178	246	4.1
E. grandis	83	202	279	4.1
E. saligna	64	167	253	3.2
E. camaldulensis	94	217	264	4.1

Results are the means of 5 replications, each replication containing 160 trees.

[a] Mean height of the 10 fattest trees per plot (160 trees/plot)
[b] Measured 10 cm above ground level.

It is anticipated that round wood products such as fence posts could be produced in about three to four growing seasons.

Measurements of water and nutrient uptake are being made and ultimately the quality of the timber will be critically examined. An economic evaluation will also be carried out following harvest of the first crop.

Distillery effluent is also being spray irrigated onto trees at Mildura. The basis purpose of this project is to devise a method of disposing of a highly offensive waste without environmental damage. Despite some initial problems the project now appears to be meeting with success.

The above trials are soon to be complemented by the establishment of a further 4 ha. trial in which the trees will be irrigated by a micro-jet system. This will enable more accurate measurements of water usage to be made.

(c) Horticulture. Use of reclaimed water for horticulture is not currently practiced in Australia. An important trial is however being carried out near Melbourne, Victoria.

A high quality, chlorinated secondary effluent is being spray irrigated onto some 2.5 ha. of vegetables. Six different application regimes are used, each replicated five times. The general layout is shown on the attached plan.

To date only one crop (lettuce) has been harvested and the results of this trial are still being evaluated. However, preliminary results indicate that yield from the plots irrigated with effluent and those irrigated with bore water are about the same. No significant difference could be detected in the leaf of the lettuce.

Marketing quality, dry matter production, heavy metal accumulation and time of maturity are still being analysed.

The agriculturalists conducting the study are extremely happy with the results to date.

Trials using effluent for the irrigation of glasshouse tomatoes, cucumbers and other crops have been carried out on the plains north of Adelaide. Preliminary results indicate that yield is about average for the district even through no fertilizers were added to the crops.

(d) Viticulture. Only two instances are known where reclaimed water is used to irrigate grape vines and in both cases the grapes are used for wine production.

On the plains north of Adelaide an average of 520 ML/a of secondary effluent is used to drip irrigate 280 ha. of vines.

Two major problems have been encountered. Firstly the lagoon effluent used at the vineyard contains large numbers of algae. The algae die in the pipeline supplying the vineyard and produce a foul smelling black sludge which can block the trickle lines. This necessitates the use of screens on the lines which then require frequent cleaning.

Secondly the effluent being used has a high salinity. To date no serious effects have been noted on the vines but there is some concern about the long term effects. It is hoped that the application of larger volumes of water will leach the salt out of the root zone.

Despite these problems good production is being achieved.

In Victoria, a small newly established vineyard near the township of Ararat, uses effluent from the local sewerage lagoons to irrigate some 15 ha. of vines. Once again a trickle irrigation system is used. The only problem encountered to date is blocking of filters on the pump. Officers of the Victorian Health Commission have examined the project and are satisfied that there are no health hazards associated with this form of re-use.

Stream Flow Maintenance

As far as can be ascertained there are no cases in Australia where reclaimed water is deliberately used for maintenance of stream flow. However, it is common practice in inland towns to discharge treated sewage effluents into local watercourses. It is important to realize that low or zero flow during dry seasons is a common feature of

Australian streams and a stream's flow at certain times may be entirely sewage effluent.

The author can see no harm in this practice which he regards as a legitimate form of re-use providing that it is done deliberately and with the necessary monitoring and controls. Problems will only arise where sewage or treated effluent "unwittingly" gains access to a watercourse.

In view of the increasing volumes of water that are being extracted from highland streams, the author believes that there will be increasing pressure in Australia to return the water to the streams after use.

Landscape Watering

Landscape watering with reclaimed water is practiced extensively throughout Australia.

The most notable example is at Glenelg near Adelaide in South Australia. The entire dry weather flow of 52 ML/day from the Glenelg treatment works is used for the spray irrigation of eight sports fields, four golf courses, a bowling green, a caravan park, parts of the Adelaide aerodrome and some 40 ha. of passive recreational land along the banks of a nearby river and the foreshore of the local beach.

The effluent used is a good quality secondary effluent that is chlorinated prior to being pumped to the various users. No problems have been experienced with this project to date and the re-use of effluent appears to have been generally accepted. It is understood that the demand for the water is so great that its use has to be rostered at certain times. Consideration is being given to providing additional storage at the treatment works so that excess winter flows may be utilized during the summer.

Many parts of inland Western Australia experience hot climatic conditions and low rainfall. Domestic water supplies may be piped to the towns in these regions, from storages located several hundred miles away. As the population of these towns increases and the expectations of the residents rises there is often a demand for grassed and pleasant recreational areas within the towns.

In order to satisfy this demand the Government of Western Australia is providing substantial assistance to towns to develop facilities to re-use effluent for landscape irrigation. This is the only case known of an Australian State or Federal Government providing direct assistance for the use of reclaimed water.

The conditions laid down for re-use are simple:-

(i) effluent should have a B.O.D. less than 30 mg/L;
(ii) effluent must be chlorinated - total coliforms in the effluent must not exceed 1,000 per 100 ML;
(iii) the effluent distribution system must be clearly identified, (i.e. special colour coding, labels on outlets, locking or keyed cocks only used);
(iv) public should be excluded from areas being irrigated;
(v) grounds should not be used for sporting purposes until they have thoroughly dried (assumed to be two to four hours after irrigation has ceased);

(vi) samples of effluent must be submitted for bacteriological and chemical analysis.

The first re-use scheme was commissioned in 1962 and there are currently some 22 towns with a total population of about 84,000 people (ranging from 750 to 12,000), that have implemented re-use schemes. The schemes have proved very popular with local people and the State Health Department of Western Australia has been unable to detect any adverse health effects related to this form of re-use.

At Woomera in far northern South Australia, the total effluent from the town's 4,000 people is used to irrigate six ovals, lawns, nature strips and thousands of trees. The effluent receives primary treatment, detention in lagoons and chlorination before re-use. Re-used effluent accounts for approximately one third to one half of the daily non-domestic water usage of the town.

The only recorded case of re-use of water in New South Wales is at Broken Hill in the far west of the State. Nearly the entire flow of 55 ML/d from the town's two treatment plants is used principally by the mining companies to establish and maintain vegetation for dust control in this arid region. Since 1942, semi-public areas such as parks and two ovals have been irrigated.

After many failures using natural water supplies, a trickle irrigation system using reclaimed water has been successful in establishing plant cover on the extensive mine residue dumps near the town.

Throughout the remainder of Australia, use of reclaimed water for landscape irrigation is restricted to isolated golf courses, parks, racecourses, etc.

However, as the conventional water resources dwindle and cost of water rises this form of re-use is becoming more common. The author believes landscape watering will be the most common form of re-use in Australia within the next ten years.

Groundwater Recharge

There are no known cases of groundwater recharge using reclaimed water in Australia.

A trial conducted near Melbourne, Victoria, is described later in this report and a major trial is about to commence near Perth, Western Australia. This second trial is aimed at determining whether a shallow aquifer could be developed sufficiently to augment Perth's limited water supplies.

Industrial Use

A great deal is said about the potential for industrial re-use of effluent. Unfortunately this potential never seems to be realized. Recycling within industries is becoming more common in Australia and this trend is likely to continue as the costs of water supply and wastewater disposal increase.

The only reported case of industrial re-use in Australia is at the Western Mining Corporation's plant at Kambalda in Western Australia. Currently about 0.7 ML/day of reclaimed water is used for reagent mixing, slurry make up water and in dust scrubbers. The main reason for use of reclaimed water is the high cost of other water in this arid region.

Municipal Use

The townships of Woomera, South Australia and Broken Hill, New South Wales, described earlier are the only known locations where reclaimed water is used for municipal purposes.

Domestic Use

There are no cases of direct re-use of water for potable purposes in Australia and it is extremely unlikely that this form of re-use will be practised in the foreseeable future.

It is the general philosophy of most organizations involved in the use of reclaimed water that such water should be used for "secondary" purposes and hence release valuable high quality water for domestic purposes.

CURRENT RESEARCH

Most of the organized research and investigation work in the field of use of reclaimed water in Australia is being carried out in Victoria.

This work is being co-ordinated, and in many cases financed, by the Reclaimed Water Committee of the Ministry of Water Resources and Water Supply. The Committee was established in 1974 and is now achieving positive results from its efforts.

The basic objective of the Committee is to ensure that the necessary information and expertise is available within the State of Victoria, so that water may be re-used for any purpose (including potable supplies) by the end of this century.

Notwithstanding this aim it is the philosophy of the Committee that reclaimed water should be used for secondary purposes wherever possible and hence release more "high quality" water for potable purposes.

The Committee believes that the general technology required to produce a high quality water is already available and hence it is concentrating on applying technology developed overseas to Australian conditions, to looking at health considerations associated with re-use of water and to carrying out feasibility studies.

The major studies are considered briefly below:-

Health Aspects

(a) Virological Studies. The first stage of this study which is designed to determine the number and types of viruses in raw sewage and effluents from various types of sewage treatment plants has now been completed. The study team at Fairfield Hospital, Melbourne, Victoria, has been able to develop specialised equipment for isolating small numbers of viruses from large samples of water, as well as developing expertise and techniques for the identification of viruses.

The results show that there are large numbers of viruses in raw sewage, maximum figures being of the order of 8,000 per litre. The types present mirror virus isolations from patients at Fairfield Hospital; a two to three week time lag followed the emergence of a prevalent virus in clinical specimens and isolation of the same virus in sewage.

Trickling filter type plants were shown to be ineffective in removing viruses. Activated sludge plants show some ability to remove viruses (which are probably concentrated in the sludge), and significant removals were recorded after chlorination of effluent from such plants.

The best results were obtained from lagoon effluent which showed no viruses. However, as only a limited number of tests were carried out, these results are not conclusive.

The investigations have now been extended to determine the reduction in virus numbers as wastewater passes through an activated sludge treatment process and an aerobic lagoon system, and the efficiency of chlorination of secondary effluent in destroying pathogenic viruses.

(b) Helminths. In Victoria we have a pre-occupation with problems of beef measles (C.bovis) in cattle, which does not seem to be of great concern to other States or Nations.

Two interesting research projects on C.bovis have recently been completed.

The first investigated the efficiency of various types of treatment plants in removing T.saginata eggs from sewage. This was determined by grazing groups of 40 head of cattle on pasture irrigated with effluent from different types of treatment plants. The cattle were then examined for C.bovis infestation. Preliminary results indicate that T.saginata eggs are able to pass through all types of plant, except those incorporating detention in lagoons.

The second was aimed at producing a vaccine to immunize cattle against C.bovis. Limited trials to date show great promise. First results using the vaccine were:-

sham vaccinated - 10/10 infected with up to 92 cysts
vaccinated - 2/10 infected with up to 7 cysts.

This work is progressing.

The incidence of T. saginata in Melbourne's population has also come under investigation. Some 4,288 patients at various institutions have been examined. Fifty-seven cases of infection with T.saginata were found, the majority (fifty-one cases) being in people of Lebanese origin. It is thought that this high infection rate is due to the custom of eating a national dish known as Kibbi, the main ingredient of which is raw meat.

(c) Bacteria. The problems related to bacterial contamination of water are well known and much investigation work has been carried out. A comprehensive report on the rate of die-off of bacteria in lagoon systems has recently been completed. This work is currently being complemented by a second study on the rate of die-off of Salmonella in lagoon systems.

Agricultural Trials

(a) Timber Growth Trials. The Victorian Forests Commission has been carrying out species trials at a number of locations for the past 3 years. Production trials are now underway at Horsham, Victoria, and Mildura Sewage Treatment Works (Victoria), and at Mildara Winery at

Merbein, Victoria.

The trials, each of which cover some 4 ha. on which are planted about 7,000 trees are aimed at determining the feasibility and economics of growing timber irrigated with reclaimed water on short rotation for the manufacture of fence posts and similar products. (See also earlier comments).

Timber growth trials are also being carried out at the Werribee Farm, Victoria. These trials are concerned with the production of timber on a commercial basis and also investigating the growth rate of some exotic and decorative trees.

A third timber growth trial is being carried out at the Latrobe Valley Water and Sewerage Board's Sewerage Farm at Dutson Downs, Victoria. An existing radiata pine plantation is being irrigated with effluent having a high industrial wastewater content, to determine the ability of the trees to renovate effluent and the effect on tree growth and soil properties.

(b) Vegetable Production. A large area has just been prepared at the Victorian Agriculture Department's Vegetable and Turf Research Station, Frankston, to investigate the growth of vegetables and turf irrigated with effluent from an activated sludge type treatment plant. (See earlier comments).

Growth trials commenced in March, 1978.

A concurrent trial is investigating the uptake of viruses, bacteria and heavy metals by vegetables irrigated with reclaimed water. This project is scheduled for completion in 3 years.

Groundwater Recharge

The Victorian Department of Minerals and Energy recently completed a recharge investigation in the Frankston area. The object of the investigation was to determine the feasibility of injecting high quality effluent into a basalt aquifer. Secondary effluent was injected onto an aquifer through a recharge bore for a period of some 6 weeks. Preliminary results are encouraging but further long term tests are required.

Feasibility Studies

A wide range of feasibility studies have been undertaken - ranging from particular situations to broad based studies.

(a) Landscape Watering in Mildura, Victoria. This study demonstated the economics and practicability of irrigating certain recreational areas (golf courses and sportsground), in the City of Mildura, with reclaimed water. It is hoped that the scheme suggested will be implemented in the near future.

(b) Agriculture in Bendigo, Victoria. The economics of transferring water, at present used for irrigation, to meet rapidly increasing urban demands, and using reclaimed water to maintain and develop irrigation is clearly demonstrated by this study. Although the study was carried out in the Bendigo area, the findings should be applicable to many similar areas.

(c) Landscape Watering of Maribyrnong Regional Park, Victoria. The practicability and economics of landscape irrigation of certain lands used for recreational purposes, (picnicking, etc.) is examined. It is suggested that the park be developed as a demonstration area to show the public the advantage of this form of re-use.

The scheme has been approved by the Victorian Commission of Public Health and is likely to commence in the near future.

(d) Strategies for Australia and Planning in Victoria. These two companion reports summarise the present status of re-use of water throughout the world, and suggest general strategies for the development of re-use of water in Australia, and more detailed proposals for Victoria.

It is anticipated that these major reports will form the basis for the development of re-use in Victoria over the next 10 years.

(e) Stream Flushing Using Reclaimed Water. Preliminary investiments into the use of South Eastern Purification Plant effluent for flushing of Kananook Creek are being undertaken by officers of the Dandenong Valley Authority, Victoria. The presence of nutrients in the effluent may preclude the use of this water because of high summer temperatures of Kananook Creek water and the possibility of algae blooms.

Advanced Sewage Treatment Processes

A considerable amount of work is currently being carried out in this area. Some of the known work that could be of significance in the future re-use of water is:-

(a) Facultative Ponds. Study into the efficiency of facultative ponds as polishing lagoons.

(b) Algae Harvesting. The feasibility of harvesting algae from high rate ponds by physical means has been studied. Such a system could not only produce a high quality effluent, but also provide a useful source of animal fodder.

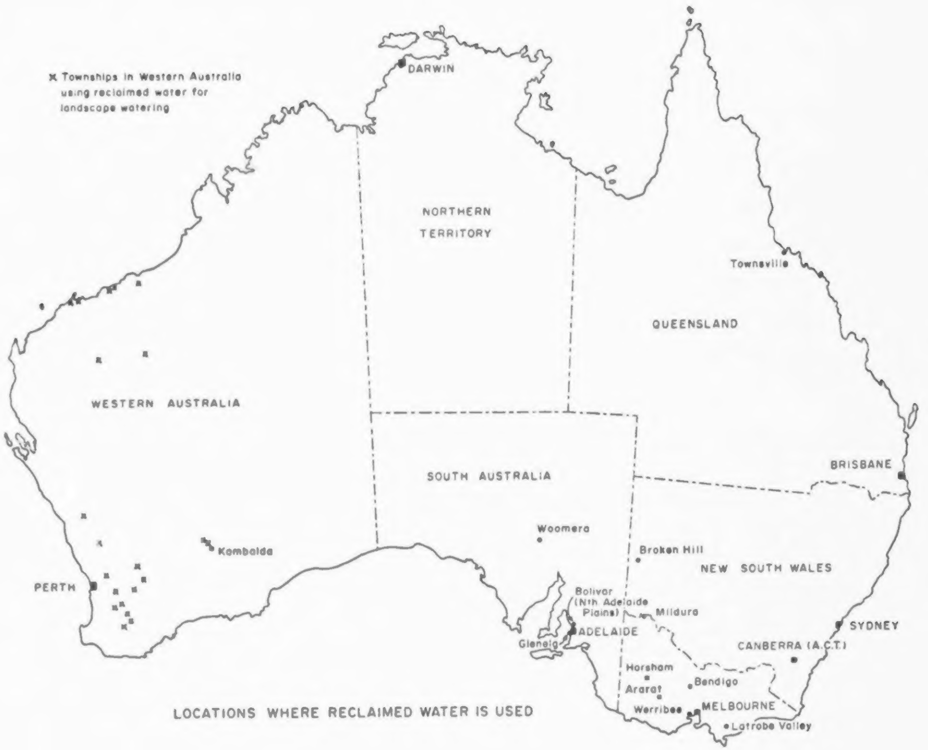
A second project is at present under way to study the removal of unicellular algae from suspensions. Two methods, one which makes use of the tendency of algal cells to aggregate under certain conditions and the other that makes use of the negative charge on some unicellular algae, are being examine. Both approaches will permit the harvesting of algae without incorporating toxic or unpalatable material, so that use can be made of this potentially valuable stock feed.

(c) Physical-Chemical Treatment Processes. In 1970, the Commonwealth Scientific and Industrial Research Organization in Australia, established a $4m^3$/hr. physico-chemical treatment pilot plant. The plant incorporates lime coagulation, ammonia stripping, recarbonation, ferric salt coagulation, filtration, activated carbon treatment and chlorination.

Recently, particular emphasis has been given to ammonia stripping by means of void towers to obviate scaling difficulties in packed towers.

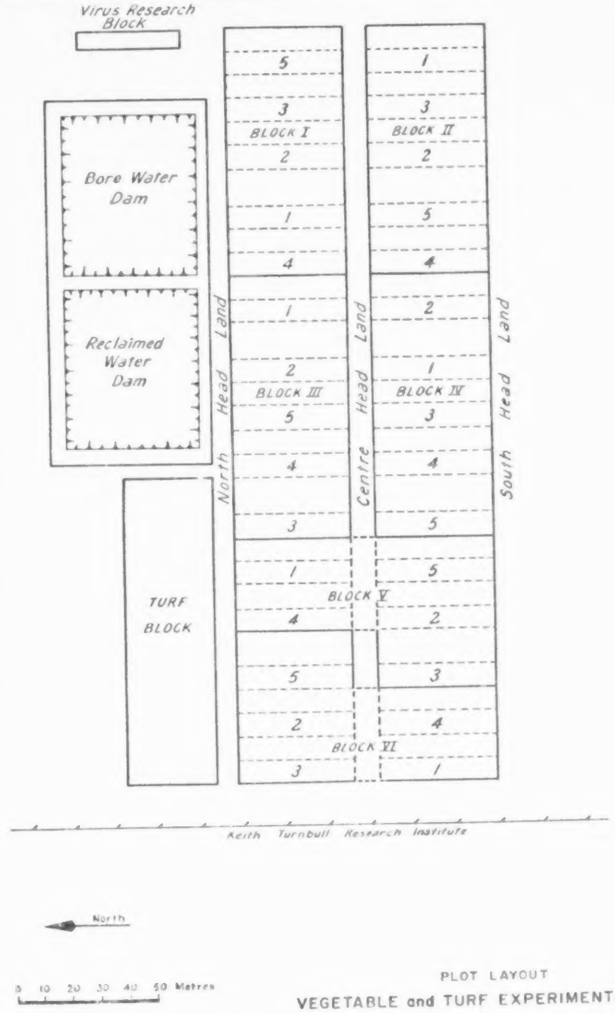
Extensive testing of desalination of the physico-chemical treated effluent using the Sirotherm process has also been undertaken.

Large scale plants will shortly be built in Perth, Western Australia.



LOCATIONS WHERE RECLAIMED WATER IS USED

PLOT	TREATMENT
1	Bore Water + Fertilizer
2	Reclaimed Water + Fertilizer
3	Bore Water + Fertilizer / Reclaimed Water (Last side-dressing)
4	Reclaimed Water + Balance Fertilizer / Bore Water (Last side-dressing)
5	Reclaimed Water + Balance Fertilizer



PLOT LAYOUT

VEGETABLE and TURF EXPERIMENTAL AREA

WASTE WATER REUSE - THE CCMS
DRINKING WATER STUDY

R. Richards
Gordian Associates, Inc.

B. G. Tunnah
Gordian Associates, Inc.

A. M. Shaikh
Gordian Associates, Inc.

K. M. Stern
Gordian Associates, Inc.

Introduction

The provision of potable water that is bacteriologically and chemically safe as well as esthetically acceptable is becoming considerably more complex, particularly in industrialized nations. As the quantity of available water resources becomes more critical due to population growth and industrialization, the availability of clean uncontaminated water for human consumption poses an increasingly serious problem.

Providing safe drinking water in the highly industrialized nations leads to considerably different problems than are usually encountered in less developed countries, where basic supply and microbiological quality are generally major concerns. For the most part, the industrialized nations have been able to control waterborne disease transmission. However, new and potentially serious questions are raised by the proliferation of industrial chemical discharges into drinking water sources, urban runoff, water polluted by human waste (both treated and untreated), and finally by the formation of new chemicals in drinking water from the interaction of disinfectant chemicals with the natural and synthetic chemicals which are commonly present in drinking water.

The NATO "Committee on the Challenges of Modern Society" (CCMS) was created in 1969 in order to bring the combined resources of the NATO membership to bear on environmental and other common problems that impact significantly on the quality of life in modern industrialized societies. The aim of CCMS is to "examine how to improve, in every practical way, the exchange of views and experience among allied countries in the task of creating a better environment for their societies...and to consider specific problems of the human environment with the deliberate objective of stimulating action by member governments."

The working methods of CCMS are based on three concepts, namely, a pilot country, orientation towards action, and open publicity. The pilot-country idea stemmed from the need to release the NATO international staff from the extensive tasks normally connected with implementing a multilateral venture. The CCMS entrusts the pilot country that has proposed a project with the responsibility for carrying it out. This includes planning the study, preparing necessary reports, and the subsequent action necessary for its realization.

National leadership of the study permits international cooperative efforts to move ahead more quickly and efficiently than under the committee approach characteristic of most of such work.

The action-orientation concept is reflected in the CCMS aim that studies be finished as soon as possible and recommendations be produced for a positive course of action by member countries and other international organizations. The CCMS has no intention of remaining actively involved in projects for five or ten years. The purpose of CCMS is to catalyze collaboration on solving problems of international concern, such as in the case of the Drinking Water Project. It was intended that CCMS should be involved in that particular project for perhaps three years, at which time the conclusions and recommendations from the study would be endorsed by CCMS and passed on to other forums for action and implementation.

The third concept, that of open publicity, is a completely new departure for NATO. The aim is to make information on the work of CCMS widely available. The CCMS meetings are open to the press and to any intergovernmental organization wishing to send observers. The meetings of experts are also open to participants from any country or international body.

In view of the existing and potential problems in the supply of drinking water which are faced by the NATO members, the U.S. submitted a proposal for a CCMS study at the end of 1976. The first meeting of experts from a total of eleven nations and two international organizations took place in Brussels in April 1977, and a series of experts meetings have been held subsequently. The next meeting will take place in Washington at the end of April 1979.

The primary objectives of the NATO-CCMS Study are to achieve a better understanding of

(1) The drinking water problems shared by industrialized countries.
(2) Solutions to these problems now available.
(3) Potential problems for which solutions are currently not available, except perhaps at prohibitive cost.

These objectives are to be achieved through a thorough state-of-the-art review of these topics, the emphasis at all times being on the latest work and information in the field.

The study is now approaching completion. Six international working groups are studying various topics related to drinking water supplies: these topics are analytical methods, advanced treatment technologies, microbiological factors, human health effects, groundwater protection, and waste water reuse. In the water reuse group, there are seven nations participating: The U.S., France, the United Kingdom, the Netherlands, Sweden, Spain and Germany. This paper describes the work of the CCMS study with respect to water reuse, and presents some of the findings of the work which has been performed by the representatives of the seven participating nations.

The subtopics which are covered by the CCMS study and are described in this paper are the following:

(1) The legislative background to the control of discharges into potential water sources.
(2) The measurement of the degree of indirect water reuse.
(3) The extent of current water reuse.
(4) Treatment options.
(5) Potential health problems.
(6) Public acceptance factors.
(7) Water resource management.

In addition, a number of case studies are described.

Legislative Background

Regulatory and Legislative Control of Discharges of Pollutants into Surface Waters

In the United States, the enactment of the Federal Water Pollution Control Act Amendments of 1972 began a comprehensive effort to "restore and maintain the chemical, physical, and biological integrity of the Nations' Waters." Similarly, other countries participating in the NATO-CCMS effort have introduced, within the past ten years, legislation to maintain or upgrade the quality of surface waters. The NATO-CCMS report is based on responses from five member nations: Germany, France, The Netherlands, Sweden and the United States. The different national programs can be compared on the following points:

- Legislative responsibility
- Administrative responsibility
- Financing

Legislative Responsibility

In three of the five reporting countries the national government establishes minimum water quality standards. These countries are the Federal Republic of Germany, France and the United States. In Germany, the Federal Government is constitutionally authorized to enact "framework legislation" to safeguard the quality of public waters. The general principle of federal legislation is to require formal authorization of any usage of water which goes "beyond customary practice," and to establish minimum quality standards as the basis upon which authorization may be granted. The major vehicle through which this is achieved in Germany is the Federal Water Act of 1957 (as amended in 1976).

Similarly, in France, the Water Law of December, 1964, establishes that all discharges "whether direct or indirect, of water or materials that might alter the quality of the surface, sub-surface or territorial waters are subject to authorization."

In the United States, the Federal Water Pollution Control Act of 1972 links the promulgation of effluent standards to objectives for ambient water quality. Like the German and French programs, United States legislation incorporates a program to issue permits for specific water uses.

In contrast, the control of water quality in The Netherlands is based on a sharing of legislative responsibility between the national and provincial governments. As in the above cases, national law (Pollution of Surface Waters Act, 1970) establishes that a license must be obtained for any water use that deviates from customary practice. However, the central government has responsibility both to articulate and to enforce water quality standards for major rivers, certain canals, the estuaries and territorial waters. Provincial governments, on the other hand, have identical legislative and administrative responsibilities for all other waterways.

The regulatory framework in Sweden has little in common with those in the other reporting nations. The Swedish system imposes no uniform water quality standards, whether national, provincial or local. Instead, it relies on a National Franchise Board for Environmental Protection, which has legal authority to grant permits for the discharge of pollutants. Decisions of this board are made on a case by case basis. The "Franchise Board" is assisted in its administrative and regulatory functions by lower level review boards which may exempt certain users from the need to apply for a permit.

Administrative Responsibility

In all five countries, any special use of water must first be authorized by a special license. The terms, duration and degree of detailed control in these licenses vary from one country to another. However, in all cases they are the most important means through which effective control is maintained over the discharge of pollutants.

The strongest national role in the administration of water pollution control occurs in the United States. The Environmental Protection Agency is authorized to issue permits to industrial and agricultural users, and may veto any permits issued by state authorities. Furthermore, it is also the principle enforcement agent, with responsibilities in the areas of inspection, review and imposition of penalties.

The Swedish system provides a strong role for the National Franchise Board, but the Board's functions are limited to the review of applications and the issuance of permits. Inspection and enforcement are shared by lower level boards, by local authorities, and by representatives of specialized agencies, none of which operate at the national level. In the Netherlands, as pointed out earlier, national and provincial functions are delineated only on the basis of the waterways for which each level of government has jurisdiction. Finally, in Germany and France, both the granting of licenses and enforcement of compliance with them, are the responsibilities of state government agencies.

Financing

The issue of how water pollution abatement programs should be financed is intimately linked to the nature of the incentives the programs offer. The principal question at stake is the use of tax incentives. Such incentives serve to discourage pollution in excess of the permitted standards and to generate revenue with which to support both monitoring and regulatory activities, as well as direct water purification efforts.

Tax incentives have the advantage of "making the polluter pay." In addition, they allow a measure of flexibility in establishing the maximum amount of permissable pollution. However, they run two important risks: first, if such taxes become an important source of revenue for abatement programs, the incentive to end the revenue-generating sources of pollution is diminished; second, they permit polluters to opt for the tax if he finds it too costly to institute pollution abatement programs -- even if the social costs of pollution are extremely high. France, Germany, and the Netherlands have all chosen to include a pollution tax as part of their regulatory systems. It is particularly important as a revenue source in the Dutch system. Neither the United States nor Sweden uses such a tax.

Measurement of Water Reuse

It is difficult, if not impossible, to measure the levels of organic compounds and pathogenic organisms found in rivers which are sources of drinking water. These pollutants are generally found in proportion to the wastewater content of the river. Therefore, in order to protect consumers of water abstracted downstream of industrial and/or municipal discharges, it is desirable to determine the degree of reuse that occurs at the point of abstraction. This is expressed by the reuse factor which is the percentage (or decimal fraction) of the river's flow which is comprised of wastewater.

Reuse can be quantified by a mass balance of the river system by using river flow, discharge and abstraction data. However, this data is not easily obtained and may not be available. In addition, the water in a river can be temporarily exchanged with groundwater resulting in less actual reuse than calculated from flow data.

Therefore, several substances are being investigated for use as indicators of municipal/industrial pollution. The criteria for an indicator substance are:

- that its concentration in municipal/industrial effluents be known;
- that it have negligible background levels in the stream;
- that it be carried freely in the river.

Several indicator substances are being investigated and are discussed by the CCMS study. Of particular interest is boron which can provide a good index of the reuse of municipal effluents since it is found at fairly constant levels in sewage effluent. The source of boron is the perborates in domestic detergents. Total organic chlorine also has been suggested as a reliable indicator of man's activity in the environment. Other indicator substances discussed include: chloride, total organic carbon and phosphates.

Although reuse indices cannot determine the toxicity of the water supply, they are valuable as measures of the proportion of harmful materials likely to be present.

Extent of Current Reuse

The direct use of water today occurs almost entirely in industry and agriculture. Industrial and agricultural reuse is an important point in any discussion of the supply of potable water since this reuse reduces the demand for natural waters which then can be reserved for potable use.

Industrial Reuse

Industry can reuse water generated internally or can accept water from an external source, such as a municipal wastewater treatment plant. Due to increasingly stringent pollution control laws, industrial wastewater is rapidly becoming too expensive to throw away following treatment. In many industries, treated effluents could be reused with little or no additional treatment. Indeed, in some cases, the quality of this treated wastewater may be better than that of the original source of supply and/or the receiving waters.

This review addresses in a general way the various water requirements of industry and the quality of water needed for each of these requirements. Different reuse systems or strategies are also identified as are the treatment systems used by industrial reuse systems.

Agricultural Reuse

The conservation of potable water supplies can also be achieved through the reuse of industrial and municipal wastewater for agricultural purposes. Agricultural reuse of water can provide the following benefits:

- low cost source of water
- economical disposal of wastewater
- effective use of nutrients in wastewater
- treatment of wastewater before it reaches the groundwater.

The principal source of water for agricultural reuse has been municipal wastewater since its composition is more uniform and it is less likely to contain toxic materials. However, the effluent from some industries can be very suitable for agricultural reuse. Besides the irrigation of croplands and pasture, municipal and industrial effluents can be used also for landscape irrigation (e.g., parks golf courses, college campuses, etc.).

This section of the report also discussed briefly some technical aspects of agricultural reuse; the effect of wastewater on soil properties and crops; and the human health concerns associated with reuse.

Treatment Options

The treatment options for two different types of potable reuse were considered in this section of the CCMS report:

- indirect reuse via groundwater recharge;
- direct reuse.

It should be emphasized that these systems for the reuse of municipal wastewater should only be depended on to supplement - not replace - potable water supplies derived from natural sources.

Groundwater Injection

Groundwater supplies can be augmented through the injection of wastewater that has undergone primary and secondary treatment. Reinfiltration provides additional treatment for the wastewater as does the buffering and dilution capacity of the underground reservoir. Care must be taken, however, that adequate treatment is provided prior to injection to prevent the degradation of the groundwater resource. The design of the treatment process should take the following into account:

- quality of untreated water;
- hydrogeology of the recharge site;
- method of recharge; and
- relative proportions of artificial and natural inputs.

Generally speaking, this treatment should eliminate suspended solids, organic matter and ammonia and reduce nitrates. In addition, activated carbon and ozonization can be used to remove and enhance the biodegradation of organic matter.

Direct Reuse

Perhaps the greatest problem associated with direct reuse is the build-up of dissolved solids in the recycled water. There are two approaches to solving this problem. The first is the semi-closed system which recycles water until the concentration of dissolved salts reaches the limits fixed by the potability norms. Water is then taken in from another source until the level of salts is reduced to the desired level. The second approach is to demineralize the water using reverse osmosis, electrodialysis, ion exchange or distillation.

The treatment train should also include activated carbon and aeration to remove organic compounds and chlorination should be used to eliminate pathogenic organisms.

Potential Health Problems

The risks to human health from the use of renovated waste water are due to the possible presence of microbiological and chemical contaminants in the water. The degree of risk rises with the degree of human contact or exposure. The Area V CCMS report examines in detail the risks associated with microbiological and chemical contaminants (inorganic and organic) in recycled drinking water.

Chemical and microbiological contaminants are examined at the levels and type of specific contaminant, known health effects, problems of detection and of elimination.

The report finds that present microbiological or chemical standards are inadequate, and that the lack of economically viable methods of detecting low level viruses or organic chemical contaminants is a serious obstacle to the use of reclaimed water. However, the most important objection against the use of renovated water for drinking is that there is, to date, insufficient information on the health risks associated with the various contaminants which may be present in reuse water.

Public Acceptance Factors

Public acceptance is a primary factor when considering any program to use renovated waste water. Increased demand for drinking water relative to existing sources of supply have heightened interest in renovated waste water as a potential source. Simultaneously, with advances in the technology of water renovation, it appears possible that quality standards can now be met as well.

Several studies of public acceptance of renovated waste water have been conducted. They reveal a number of important determinants of public attitude:

- the degree of acceptance decreases with the intimacy of contact with the reclaimed water. In the United States, less than 50% of the sample populations would accept recycled water for drinking or cooking.
- there are regional variations in the degree of public acceptance, with the greatest acceptance occurring in areas which face relative scarcity of fresh water sources.
- level of education and income appear to be positively correlated with acceptance of renovated waste water.
- the level of specific knowledge about recycled water also appears to be positively correlated with acceptance factors.

The Area V CCMS report assesses these factors influencing public acceptance, and examines methods for the successful introduction of renovated waste water for public consumption. Finally, the report compares the attitudes of the public at large with those of the scientific and medical community. The report finds that while public acceptance of renovated waste water increases with knowledge and exposure, it is precisely the most knowledgeable group -- scientists and public health officials --that has the greatest reservations about use of recycled water for drinking.

Water Resources Management

The water resources management section of the CCMS report discusses trends which impinge upon the reuse of water for potable purposes. All the NATO countries have introduced legislation concerning water pollution control/water resources management within the last 20 years and particularly since 1970.

These laws have a common feature in that all countries have organized water management at three levels: national, regional and local. As a general rule, the national level is concerned with the coordination of water policy; the regional level is concerned with systems management (particularly pollution control); and the local level is concerned with the actual management of resources, such as water purification and sewage treatment plants. In countries with a federal form of government (United States, West Germany), the trend is towards greater central government involvement in pollution control. However, in countries where the government is more centralized, the trend is to set up regional management agencies (Finland, France, United Kingdom).

There are two approaches that can be used to obtain compliance with water resources policy:

- legislative/regulatory constraints; and
- economic inducements.

The former approach requires certain actions, say the reduction of water consumption or pollution discharges by an industrial facility, to be authorized by the government. In the latter approach, penalties (pollution taxes, water charges) can be levied at a level that results in the industrial facility finding it economic to reduce its water consumption/pollution. Spain and France currently employ economic inducements to control pollution discharges.

Water quality targets are desirable so that the reduction of individual pollution discharges to a body of water can be coordinated. There are two different strategies for developing such targets. Long-term targets can be set which require a large reduction in pollution in the hope that the technology to accomplish this reduction will be developed in the future. Alternatively, less severe targets can be set, which are more realistic and are achievable on a short-term basis.

This section of the CCMS report also addresses the use of various mathematical models in the management of water resources. For instance, models can be used to control the discharges from reservoirs in order to maintain a minimum flow in a river system and thus avoid excessive reuse.

The CCMS member nations have directed most of their energies in the management of water resources towards surface water. However, it is recognized that groundwater resources will require similar attention if the quality and quantity of these sources of potable water are to be maintained. The CCMS report has identical areas that require study before comprehensive groundwater management plans and policies can be developed.

Case Studies

Many communities throughout the world draw their potable water supplies from rivers into which sewage has been discharged upstream. Although indirect reuse may have significant implications for public health, these communities are not always aware of the extent of this reuse. The Case Studies included in CCMS report review instances where indirect reuse has been documented and measured.

Ruhr Valley (West Germany)

Perhaps the best known case involving indirect reuse is the Ruhr River. The Ruhr Valley is a densely populated industrial region in West Germany. The river is used extensively both as a source of drinking water and for the disposal of treated municipal wastewater. By careful management of reservoir discharges, the reuse factor can be kept below an acceptable 22% even at extreme low flows.

Thames River (England)

The reuse of the River Thames has been quantified by both boron analysis and by using flow, discharge and abstraction data. Those studies indicate that, at the point where water is withdrawn to supply London, the reuse factor is 13 - 14% during normal flows.

Porsuk River (Turkey)

The Porsuk River in Turkey provides an example of innovative planning regarding reuse and will be addressed here in greater detail.

Eskisehir, which lies on the Porsuk River, is the seventh largest city in Turkey and has a population of 270,000 in 1975. The present water supply of the city is provided by springs; however, these sources are being used to full capacity and will not be able to meet the future needs of the city. The projected population of Eskisehir in year 2000 is greater than 750,000 and the water demand is expected to increase sevenfold between 1975 and 2000.

Obviously, the City of Eskisehir must find a new source of potable water. The Porsuk River appears to be the best available source for this additional supply. It is estimated that Eskisehir could depend on the Porsuk Reservoir, which is located on the river, for 70 million cubic meters per year.

The chief obstacle to this plan is the water quality of the river. There are several discharges upstream of Eskisehir which makes the river water unsuitable for drinking without extensive treatment. These discharges are:

- municipal sewage from the City of Kitahya; and
- effluents from three industrial facilities:
  - a fertilizer factory,
  - a slaughterhouse, and
  - a sugar factory.

The results of water quality survey showed symptoms of typical organic pollution: high levels of BOD, alkalinity, hardness, suspended solids and total dissolved solids. In addition, very high levels of nitrogen were found in the river downstream of the fertilizer factory. While the organic pollution can be treated at source by conventional means, the nitrogen problem is more difficult to solve.

Nitrogen in the Porsuk Reservoir is found in three forms: ammonia, nitrate and nitrite. Nitrite is directly toxic to humans due to its ability to disrupt the normal uptake of oxygen by hemoglobin in the blood. Nitrate is indirectly toxic since it is reduced to nitrite in the stomach. These compounds also stimulate the growth of algae in the Porsuk Reservoir which would impede treatment processes at a water purification plant. With the exception of reverse osmosis (and other membrane processes) no single treatment process can effectively remove all three forms of nitrogen. However, reverse osmosis is very expensive.

The State Hydraulic Works of Turkey examined two alternative plans for utilizing the Porsuk River as a source of drinking water.

(1) Reverse Osmosis

The first alternative would involve the removal of nitrogen compounds by reverse osmosis after water purification by conventional means at a purification plant in Iskisehir. It is estimated that this alternative would have a capital cost of $65 million and a total annual cost in excess of $16 million. A disadvantage of the reverse osmosis process is that it concentrates the nitrogen compounds (and other dissolved solids) into 25% of the incoming water while 75% is purified. The 25% must be discarded which is a significant water loss.

(2) Irrigation with Fertilizer Wastes

The second would require the discharges from the Kutahya municipal sewer system, the sugar factory and the slaughterhouse to undergo conventional sewage treatment before discharge to the river. The Eskisehir water purification plant would provide the same treatment as the first alternative, with the exception that reverse osmosis would not be included. The nitrogen problem would be solved by pumping the fertilizer factory effluent to agricultural land for irrigation. In this way, the nitrogen content of the wastewater would be used as fertilizer. This system would require lagoons to store the wastewater during the winter and a distribution network. It would cost just slightly more than $2.5 million to construct this system plus $18.6 million for the wastewater treatment plant at Kutahya. The total annual costs for this alternative would be less than $2.5 million - just 14% of the cost of the reverse osmosis alternative.

THE ROLE OF RE-USE IN HONG KONG AS A POTENTIAL FUTURE WATER RESOURCE

To-Hin LAU
Government Water Engineer/Development & Planning
Water Supplies Department, P.W.D., Hong Kong

GENERAL

Hong Kong is situated at the mouth of the Pearl River estuary within a rectangular boundary of 65 km by 43 km, within which is a mainland area together with two large islands and over a hundred small islets. The total land area is approximately 1,000 Km^2, most of which is steep and rugged. The only exception is the agricultural plain at the northern and north western part of the New Territories. There are no perennial rivers or natural lakes and water supply has been a major problem with Hong Kong since the city took shape a century ago.

WATER RESOURCES

Up to twenty years ago, the municipal water supply for Hong Kong depended solely on rainfall. Rain was collected mainly from the relatively unpolluted upland catchments with intercepting channels at the periphery which perform the function of conveying the water collected to impounding reservoirs built by damming of suitable valleys. Restriction of water supply to a few hours each day was the rule rather than exception. Importation of water from China first began in 1960 at a rate of 23×10^6 m^3/annum which has since been increased by seven-fold, equivalent to about 30% of Hong Kong's present demand.

Having exhausted the available sites for reservoirs on land, two ambitious schemes both involving the construction of impounding reservoirs from the sea together with wide-spread indirect catchments were completed in 1967 and 1978 respectively. This brought the total catchment areas up to about one-third of the overall land area of Hong Kong. The remaining areas were considered too remote to be economically feasible to develop or were polluted by habitation or agriculture. The currently developed natural catchments as well as the remaining developable catchments are illustrated in Figure 1.

The potential of ground water resource was examined in the early 1960's. It was concluded that being generally volcanic, the ground water potential of Hong Kong was not promising apart from the northern agriculture plain.

Faced with the resource shortage, a decision was made in 1971 to construct a fuel oil burning desalter of capacity 182,000 m^3/day. This desalter was successfully commissioned in 1977 and was immediately put to full production for nearly one year, making an extremely valuable contribution over the then prevailing drought. It is unfortunate that over the construction period, the price of oil went up many times, leading to a much higher unit cost of water production than originally envisaged.

With these limited resources, draw-off from the system is intentionally taken somewhat in excess of the normally accepted "reliable yield". This enables a greater yield to be available in years with favourable rainfall but leads to a deficit in the drier years. Such deficits are accepted and met by restriction of water supply to re-reduce the demand over the period of deficit.

WATER DEMAND AND SHORTFALL

The mean annual demand in 1978 was 412×10^6 m^3/annum representing a gross per capita demand of 0.25 m^3/day with approximately 98.5% of the population supplied with water. No water is supplied for agricultural use and sea water is generally used through a separate reticulation system for flushing. Historical records and medium term projections indicate a growth of demand of 6% per annum over the next 8 to 10 years. As can be seen from Figure 2 where the resource availability is superimposed onto the demand projection, the currently available water resources will become fully committed by 1984. Beyond that time, a substantial quantity of new resources, estimated to be of the order of 35×10^6 m^3/annum increase each year, will be required to meet the growth in demand. This is a formidable task and one of the potential sources of additional supply is the re-use of water.

HISTORY ON RE-USE IN HONG KONG

Back in 1965, the feasibility of re-use of water as one of the potential future resources was examined. At that time, the population was concentrated in the established urban areas. The domestic sullage water from buildings and the waterborne toilet discharge were mixed together prior to being collected by the urban sewage system. Sewage disposal was effected by direct discharge into the harbour with or without screening through a number of outfalls. With sea water used for flushing, the sewage was brackish with salinity up to 7,000 p.p.m. NaCl. The prevailing restricted supply together with the generally fairly low standard of living led to sewage of higher concentration than normally found in urban development elsewhere. The lack of facilities and technology to produce reclaimed water of acceptable quality, the general psychological reluctance to re-use and the availability at that time of other reasonably economical alternative resources led to the deferment of further investigation into re-use for over a decade.

RECENT RE-ACTIVATION

With the forecast shortfall in water resource in the near future, the diminishing area of developable catchment and the very much higher cost of desalting sea water, the need to take a fresh look at the feasibility of re-use of water arises once again. There are two major sources of water from which fresh water may be reclaimed for re-use. The first source is the water in the lower reaches of streams and rivers which are generally grossly polluted. The main causes of pollution are the intensive agricultural activities which include pig and poultry raising, human habitation and in many places, discharge from small scale industries. These streams and rivers together with any wells available normally provide water for agricultural needs and at the same time act as the drainage system for the return water and discharge from all the activities within their catchments. Except during heavy rain, the quality of such water is entirely unsuitable as a water resource without vigorous treatment. The potential yield of such polluted streams in the northern and north-western agricultural plain as illustrated by Figure 1 is estimated to be 50×10^6 m^3/annum which, if collected, could form a significant contribution to the water resources.

The other source is the treated effluent from the sewage treatment works in the new towns. Since the early 1970's, the Government policy has been the decentralisation of urban growth and four major self-contained new towns and several rural and market towns capable of accommodating a total of over two million people are being developed. Centralised collection and treatment of sewage and waste water is part of the basic infra-structure to be provided and for those towns draining into rivers and semi-enclosed sea inlets, the waste-water will be treated to secondary standard (20 p.p.m. B.O.D. and 30 p.p.m. S.S.) prior to discharge. Such a requirement is determined by environmental consideration although the effluent would provide an invaluable source for the application of recently developed water reclamation technology to turn it into a re-usable source of municipal supply. The previous problem of saline sewage could be overcome by the use of treated effluent instead of sea water for flushing and the same system could provide water for the necessary amenity planting. The quantity of effluent available would increase with the growth of population and it is estimated that upon full development, a foreseeable potential contribution of $140 \times 10^6 m^3$/annum could be expected with the ultimate potential following the increase in water demand.

While the potential for water reclamation has increased as a result of improved technologies, there still remains the need to prove the physical and hygienic suitability of the product obtained by such processes. Another major obstacle is to convince the public to accept re-use and overcome the not-unnatural psychological resistance in relating fresh water supply to foul discharge. The objective of saving water through re-use is further complicated by the layout of the urban areas in having light industrial and commercial zones intermingled with domestic zones, thus rendering the implementation of a separate system feeding reclaimed water to specific requirements complicated and expensive.

EXPERIMENTATION PROGRAMME

With all these problems to be overcome, a very carefully planned approach is therefore necessary.

Initially, re-use of treated effluent is to be introduced for flushing and amenity planting. For flushing, a pilot trial on the use of secondary effluent has proved to be successful. The experiment is now extended to supply the flushing demand of a large public housing block with a population of about 5,000. The secondary effluent supplied will be chlorinated and if found necessary, clarified with coagulant and tinted with colouring matter for distinction. In respect of amenity irrigation, the planting of turf, flowers and trees in the nearby race-course has been successfully completed using secondary effluent without further treatment.

Being in the tropics, the demand for air-conditioning is very high in Hong Kong and this in turn has led to the need for water as a heat rejection medium. The scarce water resource in Hong Kong does not permit the use of municipal fresh water supply for such purposes, even through an evaporative cooling tower, unless the air-conditioning requirement is essential to an industrial process and no other practical alternative sources are available. This restriction has led to the use of sea water, sometimes involving long lengths of delivery pipelines or air cooled units. The alternative available would be the use of treated effluent via the distribution system for flushing and amenity irrigation and an experiment consisting of a small commercial evaporative cooling tower and heat exchanger with an artificial heat input is in the final stage of preparation. It is the intention to establish the quality of effluent required for satisfactory operation of the heat exchanger and cooling tower and to identify the effect and extent of corrosion and scaling. The need may be revealed for further in-house treatment of the effluent prior to being used in the system.

All these experiments are in various stages of progress and it is anticipated that initial conclusion would be reached in six months' time. The results of these applications, particularly on flushing, would be extremely important to the future of re-use in Hong Kong. If successful, it would assist in bringing public acceptance toward the more sophisticated application of re-use and it would provide a treated effluent without sea water contamination, the removal of which would otherwise be very expensive through reverse osmosis.

WATER RECLAMATION PILOT TESTING

The above-mentioned experiments are only for low-grade re-use or, in the case of flushing non-consumptive re-use. In terms of the proportion of sources available, this application as an additional water resource would be very limited. The real potential would lie in the re-use of available polluted stream water and secondary effluent after proper treatment as a source of municipal supply. In order to establish the advanced wastewater treatment processes necessary, the quality of the reclaimed product water and the associated costs and problems of future production, a consultant in this field was appointed in December 1978 to carry out the necessary feasibility study through the operation of a pilot test plant in Hong Kong.

The pilot test plant being purchased will consist of three test trains as illustrated in Figure 3. The first train will be used for treatment of secondary effluent. It will consist of chemical flocculation and clarification under which phosphorus removal will take place. The flocculants to be experimented will be lime, alum and ferric chloride. The clarified super-natant will be bi-furcated into two trains. One train will test for physical removal of ammonia through air stripping under high pH and then re-carbonation to reduce the pH. The product will be put through a dual media filter to remove any remaining suspended solids. The other train will test for the fixed growth nitrification using a rotating biological contactor then fixed growth denitrification via suitable filters.

Another train will be for the testing of primary effluent or grossly polluted river water. The feed water will be put through an activated sludge process with air supply and retention adjusted to give complete nitrification of the ammonia present. Phosphorus removal will be carried out similar to the first train after which fixed growth denitrification filtration will be applied.

The product water from all three trains will be put through a polishing system where the water can be chlorinated, ozonated and treated through granular activated carbon adsorption.

The entire pilot plant will be mounted on four skids to be connected by flexible hoses in order to provide mobility and flexibility in testing. Ample measuring and sampling points will be provided to enable the achievement at various stages to be measured. A mobile laboratory will be attached to the pilot plant to carry out most of the day-to-day straightforward testings leaving the more complicated tests using precision apparatus such as the atomic adsorption spectrometer and total organic carbon detector to be carried out in the main laboratory. Included in the test programme will be the examination of bacterial and viral content in both the feed water and product water. Bacteria will be examined by the conventional method using E. Coli count as an indicator. The testing of viruses will be carried out by the Pathological Unit of the Medical & Health Department of Hong Kong Government with initial concentration and elusion to be carried out by field staff.

Table 1 gives a typical analysis of the secondary effluent and polluted stream water to be subjected to advanced waste water treatment pilot tests. While the pilot testing will establish the quality attainable and the process and costs involved, the degree of treatment under full scale application will very much depend on the approach to be followed in actual application as well as the existence of other forms of treatment before or after the advanced treatment processes to be employed.

The programme of the testing is tight in view of the imminent shortfall requiring initial decision to be made by early 1980. Tenders for the purchase of the pilot plant are expected to be received by the end of March 1979 and the plant should be commissioned in Hong Kong by August 1979. The first test site will be adjacent to a pilot sewage treatment plant producing the necessary secondary effluent and a nearby stream which is grossly polluted. The testing programme envisages initial conclusions to be drawn by February 1980 and the testing will continue for another 6 to 9 months to refine the initial findings as well as to provide data for design and costing of a full scale plant should the process prove feasible. The pilot plant will then be moved to other sites to establish the feasibility of treatment of other polluted sources as it is anticipated that the types and degree of pollution will vary significantly between various localities.

THE APPLICATION OF RE-USE

Re-use is not something new. In fact it has been practised since mankind exist, in the form of repetitive use of water by inhabitants along the rivers. Such river water taken from selected abstraction points and treated conventionally are quite acceptable when used as a municipal and potable supply. It would be logical therefore for a natural link to be introduced into the artificially or deliberately created re-use cycle. Apart from meeting the traditional psychological acceptance, such a link would enable the reclaimed water to be mixed with natural sources and enable natural purification to take place prior to being drawn off to supply.

Hong Kong unfortunately has no facilities such as natural rivers and lakes. It would therefore be necessary to create similar natural cycles by blending the reclaimed water with natural yield and storing the same in either existing or newly constructed impounding reservoirs. Through suitable arrangement for inflow, mixing and the choice of draw-off positions, such reservoirs would perform similarly to natural lakes and rivers to provide the necessary retention period and the natural purification. The introduction of such natural links would add significantly to the cost of re-use, particular if new impounding reservoirs were to be constructed for the purpose. With sites suitable for new water schemes become more and more difficult to find, any proposal for a new impounding reservoir would give rise to considerable social and environmental complication which would ultimately reflect in the cost of the schemes.

INDUSTRIAL IN-HOUSE RE-USE

Limited by the inter-mingling of industrial zones with other land uses, it is not economically feasible to provide a supply of reclaimed water to meet industrial demand only. Even if such a system were introduced, industrialists would be very cautious and reluctant to use such water which might contain traces of impurities that could affect their processes or products. This would induce limited and low grade applications such as washing-down, cooling and dilution of discharge etc. and as most trades involve light and processing industries, the demand would not be high.

On the other hand, in-house re-use, if it proved to be economically feasible, could be applied more confidently as the nature of the impurities would be known to the industrialists. Any form of re-use would involve additional processes and handling and the costs would depend on the percentage or sophistication of re-use aimed at. All re-use processes would involve land which is an expensive and scarce resource in Hong Kong. The degree of such in-house re-use would depend on the available economic incentive. The cost of municipal supply to industries is extremely moderate at US\$0.18/m^3. The recently proposed revision of tariff will raise the charge to US\$0.23/m^3. Such water charge will obviously not provide a great incentive for re-use except possibly for the very large water consumers such as bleaching and dyeing. Nevertheless, Hong Kong Government is encouraging any effort towards re-use in proposing that any industrialist who can demonstrate the use of water is essential and the water charge is a significant part of their production cost and who has taken steps to reduce the consumption through re-use may be given a concessionary rate of supply for an interim period. Admittedly, the incentive is still only marginal. Both increased water tariff and this concessionary arrangement may still not be sufficient to provide adequate motivation for re-use. A levy on polluting discharges would provide additional motivation, but any such levy would undoubtedly lead to strong opposition.

CONCLUSION

While there are very limited examples, both in number and in period, throughout the world for successful application of re-use as a source for municipal or potable supply, the future water supply problem to be faced by Hong Kong justifies an in-depth exploration of this potential source. It is proposed to establish the technical and economical feasibility as soon as possible and to this end, pilot testing will shortly be put in hand. It is hoped that the findings would contribute to and stimulate other similar works in the world.

ACKNOWLEDGEMENT

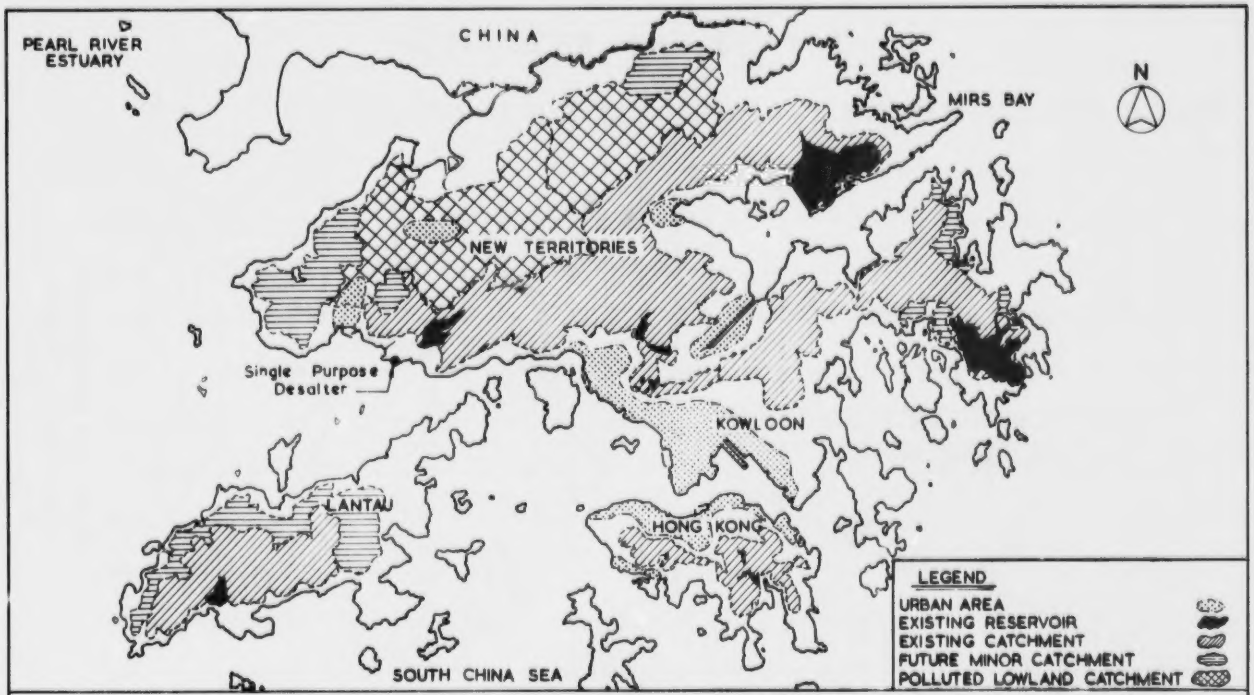
The author wishes to take this opportunity to thank Mr. W.D.A. Tucker, M.A., F.I.C.E., F.I.W.E.S., J.P., the Director of Water Supplies, Public Works Department, Government of Hong Kong for his kind permission to the presentation and publication of this paper.

TABLE 1

Summary Of Quality For Period June 78 - Feb. 79

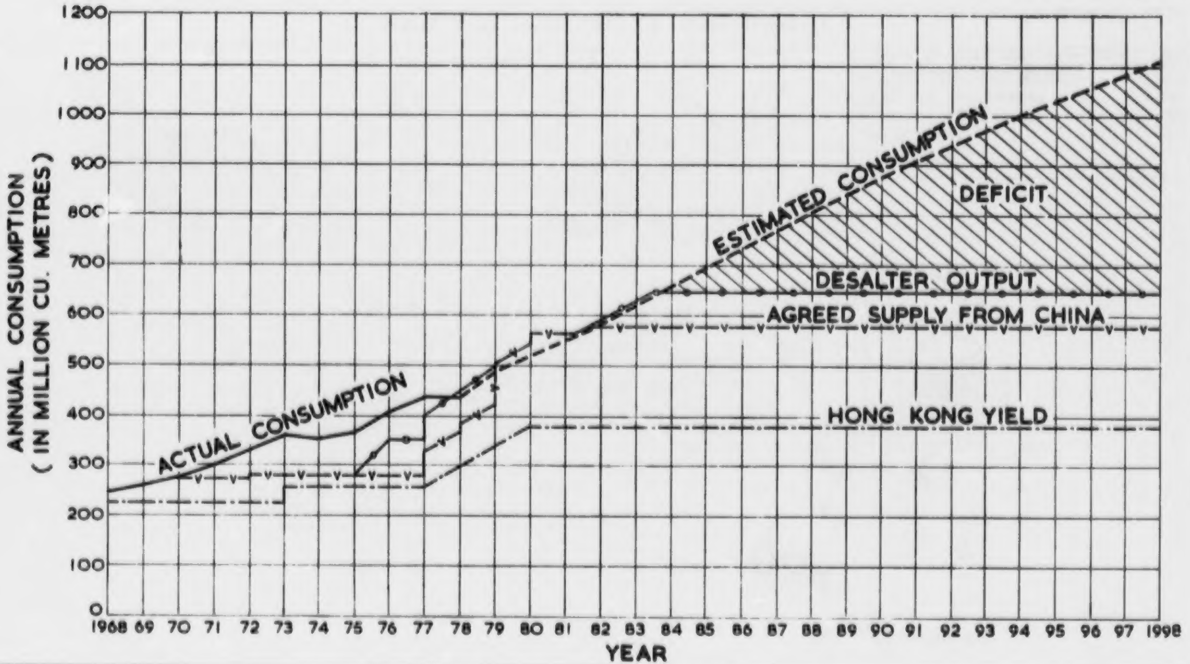
(Chemical Results Expressed In mg/litre)

	Polluted Stream Water				Secondary Effluent			
	No. of Tests (Approx)	MAX.	MIN.	AV.	No. of Tests (Approx)	MAX.	MIN.	AV.
pH	20	7.2	6.7	6.9	23	7.5	6.7	7.2
Colour	20	350	20	154	23	60	15	36
Turbidity	20	135	13	43	23	50	3.7	12.4
Conductivity	20	1103	131	469	23	2749	715	1049
Ammoniacal N	20	42.8	1.53	17.7	23	34.3	5.3	22.4
Albuminoid N	20	3.75	0.43	1.80	23	4.9	0.5	1.2
Nitrite N								
Nitrate N	20	2.48	0.06	0.61	3	4.92	2.43	3.80
Oxygen Absorbed Value	20	38.8	3.6	14.4	23	9.75	3.75	5.82
Suspended solids					23	72	4	20
T.D.S.								
Residual chlorine								
Alkalinity($CaCO_3$)	20	315	24	139	19	200	34	139
Total hardness ($CaCO_3$)	20	142	30	84	19	281	66	116
Calcium (Ca)	20	36.0	9.2	22.9	9	30.2	20.0	25.0
Magnesium (Mg)	20	15.9	1.7	6.5	9	43.4	3.7	11.2
Chlorides (Cl)	20	176	12	60	20	802	121	234
Sulphates (SO_4)								
Phosphates (PO_4) ortho	20	43.27	3.03	18.96	23	27.54	3.93	15.36
total	20	52.72	9.44	23.01	19	44.26	4.36	23.50
Fluorides (F)								
Iron (Fe)	20	5.00	0.95	2.25	7	1.42	0.08	0.41
Manganese (Mn)	20	1.37	0.04	0.51	8	0.29	0.06	0.24
Aluminium (Al)								
Silica (SiO_2)	20	22.7	4.8	15.4	3	15.6	9.5	12.5
B.O.D.	20	122	3.4	44.6	23	46.5	5.50	17.53
C.O.D.	20	218	37	108	15	223	13	116



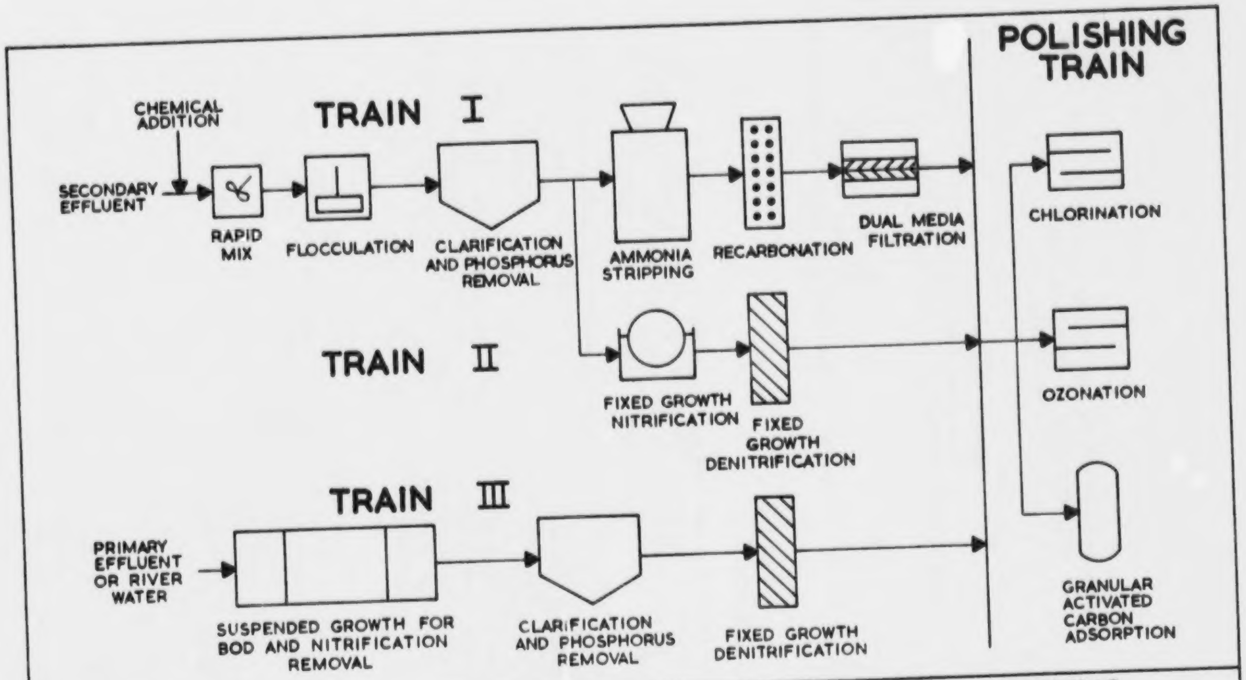
EXISTING AND DEVELOPABLE CATCHMENTS

FIGURE 1



RESOURCES AND DEMAND

FIGURE 2


POLISHING TRAIN
TRAIN I
CHEMICAL ADDITION
SECONDARY EFFLUENT
RAPID MIX
FLOCCULATION
CLARIFICATION AND PHOSPHORUS REMOVAL
AMMONIA STRIPPING
RECARBONATION
DUAL MEDIA FILTRATION
CHLORINATION
TRAIN II
FIXED GROWTH NITRIFICATION
FIXED GROWTH DENITRIFICATION
OZONATION
TRAIN III
PRIMARY EFFLUENT OR RIVER WATER
SUSPENDED GROWTH FOR BOD AND NITRIFICATION REMOVAL
CLARIFICATION AND PHOSPHORUS REMOVAL
FIXED GROWTH DENITRIFICATION
GRANULAR ACTIVATED CARBON ADSORPTION

ADVANCED WASTE WATER TREATMENT PILOT TEST TRAINS

FIGURE 3

RECLAMATION OF WASTEWATER TREATMENT PLANT EFFLUENT FOR HIGH QUALITY INDUSTRIAL REUSE IN SAUDI ARABIA

Anton A. Kalinske
Vice President
Camp Dresser & McKee Inc., Walnut Creek, CA

John F. Willis
Vice President
Camp Dresser & McKee Inc., Boston, MA

Stephen R. Martin
Project Engineer
Camp Dresser & McKee Inc., Boston, MA

A. PROJECT BACKGROUND

The General Petroleum and Minerals Organization (Petromin) of Saudi Arabia is presently expanding the capacity of its refinery in Riyadh to process an additional 100,000 barrels of oil per day. Completion of the expansion project is scheduled for September 1980.

Plans to meet increased water requirements for the refinery expansion through the use of groundwater from a nearby well field were rejected by the Saudi Ministry of Agriculture and Water because of water scarcity in the area and the potential adverse effect on the groundwater level. Reclaimed effluent from the Riyadh wastewater treatment plant (WTP) was suggested by the Ministry as an alternate source for the refinery's expanded needs. Subsequently, Petromin petitioned the Ministry of Agriculture and Water, and obtained permission to utilize 20,000 m^3/day of Riyadh wastewater effluent after reclamation in the Refinery. Water from an existing shallow well will remain the source for drinking purposes.

To meet additional requirements of the refinery expansion, Foster Wheeler Italiana (FWI) was selected to prepare the design, and to procure equipment for the Crude Stabilization and Wastewater Reclamation Facilities (WRF). The environmental engineering firm of Camp Dresser & McKee Inc. (CDM) was retained by FWI to provide consulting services related to the design of the Wastewater Reclamation Facility of the refinery expansion project.

Existing Refinery Water System

Currently, well water, having total dissolved solids (TDS) of approximately 1,600 mg/l, is treated in a two-stage electrodialysis unit to reduce the TDS to approximately 650 mg/l for use as potable and cooling tower makeup water. This water is then further treated by ion exchange to reduce TDS, hardness, and silica concentrations to acceptable levels for boiler feed water. Potable water is chlorinated and stored in an elevated standpipe. The refinery has a present maximum process water demand of approximately 500 gpm, of which 300 gpm is for boiler feed water. Potable water demand is estimated at 20 gpm. The water demand to be met by the WRF includes the water requirements for both the existing refinery and the new refinery expansion. Normal and maximum future water requirements are summarized in Tables 1, 2, and 3.

Water requirements at the refinery can generally be classified in three quality levels as follows:

1. Utility water for hose station and fire water characterized by:
    - Low suspended solids;
    - Biological and chemical stability for storage in steel tanks without causing corrosion problems and odors due to anaerobic biodegradation of organic matter;
    - Positive Langelier Index;
    - Non-pathogenicity in case of body contact.
2. Intermediate quality water for cooling tower makeup and desalting (refinery process) use:
    - Total dissolved solids concentrations less than 500 mg/l.
3. Boiler feed water:
    - Electrical conductivity less than 20 micromhos/cm;
    - Silica content less than 0.5 mg/l.

The low total dissolved solids requirement, as indicated by the low electrical conductivity and silica, is to minimize scaling and blowdown in boilers used for generating 600 psig steam.

Riyadh Water and Wastewater Quality and Treatment Systems

The city of Riyadh presently receives its water supply from two major sources: 70 percent from groundwater sources, and the remainder from local surface water.

The supply is currently processed through three existing water treatment plants with two new plants under construction. Total dissolved solids for the treated water ranges from 800 to 1,200 mg/l, with an average of between 900 and 1,000 mg/l. Reverse osmosis facilities are under construction and/or planned at all five plants to reduce the TDS to approximately 500 mg/l.

A wastewater collection system, presently serving 20 to 30 percent of the population, conveys wastewater to the WTP located approximately 7 kilometers south of the city. An extensive program of domestic connections is being pursued to incorporate unserved areas into the system. Influent flows to the WTP average slightly more than 40,000 m3/day, with occasional peak flows of 60,000 m3/day. Average daily wastewater flows are expected to reach 80,000 m3/day by 1980 and 160,000 to 200,000 m3/day by 1990.

The WTP is a two-stage trickling filter facility with a design capacity of 40,000 m3/day. A recent report for expansion and modification of the plant calls for a doubling of plant capacity by converting the process from two-stage to single-stage filtration. To achieve an overall 80 percent biochemical oxygen demand (BOD) removal, aerated lagoons will be added to polish the plant effluent prior to disposal.

Riyadh wastewater differs from typical U.S. domestic wastewaters in the high concentrations of inorganic constituents. The mean value of the wastewater influent TDS is approximately 3,000 mg/l. Table 4 summarizes available WTP effluent quality data.

Relationship of Riyadh Water and Wastewater Qualities

The inorganic constituents of wastewater are generally determined by the quality of the water supply. Normal domestic use results in an added increment of 200 to 300 mg/l of total dissolved solids to the water. However, in the case of Riyadh, the added total dissolved solids increment is approximately 2,000 mg/l. Possible sources contributing to these high TDS levels are:

- Brines discharged from water softening facilities in the Riyadh area;
- Infiltration of groundwater with a TDS of 5,000 mg/l;

- Salts in the soil may be dissolved by septic tank leachates, thus contributing to the high TDS of the groundwater. The soils in the Riyadh area are derived from limestone and contain water soluble sulfates;
- Low flows and frequent blockages in the collection system combined with high temperatures are conducive to hydrogen sulfide production which can attack concrete sewers, causing calcium salts to enter the wastewater.

B. WASTEWATER RECLAMATION FACILITY: STUDY OF ALTERNATIVES

Wastewater Quality

Because of the wide fluctuation in quality observed in the available data and the uncertainty in predicting future conditions, it was decided that the design parameters to be utilized in the WRF design should be based on primary effluent quality from the WTP. While this is a conservative approach, it does suggest a design which will be somewhat independent of the biological treatment efficiency of the WTP, especially in regard to BOD removal. It is anticipated that a better quality secondary effluent will result in a more reliable operation, lower chemical consumption, and a better quality product.

The design criteria are generally based on use of the median value of analyses as the normal value on which chemical consumption and operating costs are based. The 10 or 90 percentile values were, in most cases, taken as the worst conditions for determining the design range. The WRF's influent characteristics to be used for design parameters are listed in Table 5.

Alternative Unit Processes

The general criteria and assumptions used for development of the process alternatives are as follows:

- Reclaimed water will not be used as potable water;
- All treatment processes and mechanical equipment shall consist of a sufficient number of installed units arranged in parallel so that rated capacity can be provided with any single process unit or equipment item out of service for repair or maintenance;

- Disposal of reject water from the water reclamation facility will be by evaporation in the refinery's existing evaporation pond;

- Four existing tanks, each at 79,510 m3 capacity, will be used to store utility water. This provides for approximately 30 days' storage, based on normal refinery water demand. Storage is also available for cooling tower makeup and desalting (refinery process) water, as well as boiler feed water;

- The WRF will be located at the refinery site. Land is not readily available at the Riyadh WTP. Also, Petromin can maintain better control over the operation of the facility;

- Because of the high cost of chemicals in Saudi Arabia, cost-effective processes which reduce consumption or permit recovery of chemicals were given priority. Lime recalcination, magnesium salt recovery for coagulation, and activated carbon regeneration are three examples.

Treatment alternatives were considered in two parts: (1) the pretreatment processes preceding demineralization; and (2) the demineralization processes. Treatment goals preceding demineralization are intermediate in nature and are established by pretreatment requirements of the demineralization process and the established requirements for fire and utility water. The pretreatment processes must be capable of treating the widely variable influent wastewaters to a consistent quality level so that the demineralization processes will operate at maximum efficiency to achieve the effluent quality goals on a reliable basis. The goals of the pretreatment processes are shown in Table 6.

Table 7 lists the unit processes considered for the WRF. A description of these processes follows.

Pretreatment Processes

Grit Removal:

Grit removal is considered a necessary unit process, since the nature of the sandy environment allows silica particles to enter the process stream where there is exposure to winds. Grit in the process train becomes a potential problem when chemical reclamation and reuse are considered. Grit would tend to concentrate in the reclaimed sludges. Removal is thus considered necessary in order to separate grit materials in the pretreatment processes.

Coagulation/Softening/Sedimentation:

Chemical coagulation and sedimentation represent the most attractive alternative for suspended solids removal. The use of chemical coagulation and sedimentation in post treatment of conventional biological secondary effluents is a long-established wastewater treatment practice. The principal coagulation chemicals are aluminum salts, iron salts, and lime.

Lime is used as the principal chemical for treatment of the wastewater because of its proven capability to reduce the soluble carbonates and react with soluble organics to reduce BOD and COD.

Due to the high cost of chemicals in Saudi Arabia, chemical reclamation is justified. Seventy to 80 percent of the lime sludge can be recovered and reused. Lime has the added advantages of providing an effective disinfection process due to the high pH and the resulting sludge has good dewatering characteristics. High lime treatment permits precipitation of magnesium hydroxide, which is an excellent coagulant.

pH Adjustment:

Adjustment of the pH from 11.0 - 11.5 downward following lime treatment is necessary because most membrane demineralization processes operate at pH levels below neutrality and also to prevent further precipitation of calcium carbonate. The required downward pH adjustments can be effected by adding an acid or carbon dioxide (recarbonation). Since lime recalcination is employed, carbon dioxide will be a readily available byproduct of the recalcining furnace stack gases, though some capital expenditures will be necessary for compression and transmission of the gas.

Cooling Towers:

Influent wastewater temperatures to the WRF can be as high as 40°C. This temperature can be expected to increase through the pretreatment processes, especially when the water is stored in the steel tanks. Temperatures resulting from long detention times in steel tanks could reach as high as 50°C. This temperature cannot be tolerated in membrane-type demineralization processes. For membrane processes, it is thus necessary to provide cooling in the pretreatment process train. Cooling to 2°C above wet bulb temperature can be economically accomplished by induced draught cooling towers.

Filtration:

Granular media filtration is proposed as the unit process for final particulate removal. Particulate matter remaining after lime treatment includes colloidal material, biological matter, and some chemical precipitates. To improve the filterability of these particulates, a small dosage of a metal ion coagulant and/or polymer is added to the filter influent to increase floc size and to strengthen floc shearing resistance. Filtration can be achieved in either pressure vessel or gravity configuration.

Activated Carbon:

Activated carbon is proposed for removal of soluble organics. Upon contact with a wastewater containing soluble organic material, granular activated carbon selectively removes various organics by adsorption. The effectiveness of adsorption rapidly decreases above a pH of 9.0, thereby requiring pH adjustment prior to activated carbon contacting.

Activated carbon regeneration can be accomplished by solvent, acid or caustic wash, steam reactivation, or thermal regeneration. Due to the undesirable byproducts of the wash systems, thermal regeneration is the most common approach. Thermal regeneration was chosen for its proven regeneration capability, relatively low energy costs (in Saudi Arabia) and similarity with the methods available for lime recalcination.

Demineralization Processes

Electrodialysis:

Electrodialysis is the oldest of brackish water desalting processes employing a membrane. The process, first conceived in 1940, became technically feasible in the early 1950s when techniques were developed for producing ion-selective membranes. Ion-selective membranes are essentially ion-exchange resins in sheet form. The electrodialysis process involves subjecting a stack of cation and anion selective membranes to the passage of a direct electric current. The current acts as the transport mechanism for the ions across the membranes. Typical selectivities are greater than 90 percent. Electrodialysis provides about 85 percent water recovery, however, it is highly sensitive to dissolved organics. Unless organics were removed to negligible levels, significant maintenance problems could be anticipated and the potential for fouling a downstream anion resin would be increased.

Based on the wastewater quality influent to the WRF, it is estimated that capital and operating costs for electrodialysis and reverse osmosis would be comparable. However, recent international bidding has shown reverse osmosis to have the competitive edge.

Reverse Osmosis:

Reverse osmosis has several advantages over electrodialysis, namely, the capability of removing COD, BOD, silica, ammonia, particulates and certain other organics. Reverse osmosis provides a barrier between the feed wastewater and the product water, enhancing its use for boiler feed.

Reverse osmosis was the subject of laboratory studies in the 1950s but was not commercially developed until the early 1960s when membranes were introduced that could reject salts adequately and simultaneously produce the flux rates needed for a practical process. In principal, reverse osmosis involves the placement of a semi-permeable membrane between the concentrated solution and the demineralized water or dilute solution. The process of reverse osmosis operates by increasing the pressure on the salt solution above the osmotic pressure, thus forcing the flow of liquid through the membrane. The salts in the solution cannot pass through the membrane.

Reverse osmosis is highly effective at the total dissolved solids concentration considered for this project, about 2,450 mg/l. The major operational expense associated with reverse osmosis is the power required to produce the hydraulic pressure on the feed solution to force the water through the membrane against the osmotic pressure. While reverse osmosis can tolerate some dissolved organics, which are rejected, it is extremely sensitive to turbidity, temperatures in excess of 35°-40°C (depending on membrane type), colloidal fouling, and carbonate, sulfate and silica scaling.

Calcium carbonate and calcium sulfate are among the most common scaling salts encountered in natural waters and are the most common cause of scale in reverse osmosis system. In reverse osmosis, as product water is removed from a feed solution containing calcium and sulfate, the concentrations increase. If the limits of saturation are exceeded, precipitation of calcium sulfate and calcium carbonate will occur. Compared to calcium carbonate, calcium sulfate is relatively soluble in water. Therefore, for normal waters, the saturation limit for calcium carbonate will be reached first. Calcium carbonate scaling is reduced by the addition of acid. Calcium sulfate scaling is inhibited by the addition of sodium hexametaphosphate. This precipitation inhibitor represses both calcium carbonate and calcium sulfate by interfering with the kinetics of crystal formation. The amount of acid and sodium hexametaphosphate required can be reduced by softening the supply feeding the reverse osmosis process.

Based on the WRF water supply, water recovery will be limited to a maximum of 75 percent because of calcium sulfate saturation in the reject stream.

Reverse osmosis systems vary in configuration. This is evident in the various hollow fiber, tubular, and spiral wound designs. Spiral wound configurations have the most operating experience for wastewater feeds.

Ion Exchange:

There are several ion exchange systems that can be used depending on the quality of the feed and the final effluent composition desired. One of the systems is the two-bed demineralizer, which consists of a cation exchange resin (in the hydrogen or acid form) in the first bed and an anion exchange resin (in the hydroxyl or base form) in the second bed. The water to be demineralized passes through the cation exchanger where the cations of the dissolved salts are exchanged for hydrogen. The strongly acidic effluent from the first bed is passed through the anion exchanger where the anions are replaced with hydroxyl groups. These in turn combine with the hydrogen ions to form water and a neutral product. In this system, the cation exchanger is normally regenerated with an acid (e.g., sulfuric) and the anion exchanger is regenerated with an alkali (e.g., sodium hydroxide), to restore the beds for further demineralization. Use of rinse water to remove excess regenerants is a normal part of the regeneration step.

Among the advantages of ion exchange is that the process is basically simple and easy to operate; capital investments are comparatively low; the process runs smoothly, and is capable of complete demineralization; and the water recovery is relatively high (in excess of 90 percent). Although ion exchange resins are vulnerable to organic fouling, they are less affected by iron and manganese contaminations, and do not require the pH adjustment of the feed water as do the membrane processes. The chief disadvantages of ion exchange demineralization are that costly regenerants are required and the amounts of regenerants is proportional to the salts removed, which limits the total salinity that can economically be removed from a brackish water. These characteristics indicated that ion exchange was more suitable as a finishing stage for boiler feed water production after the bulk of the salinity and residual organics were removed by another process.

Vapor Compression Evaporation:

The vapor compression evaporation process is a recently developed distillation method in which the influent water is adjusted with acid or caustic and then pumped through heat exchangers which bring its temperature close to its atmospheric boiling point by recovering the heat contained in the hot product-stream from the evaporator. A partial vacuum is applied to the hot water which lowers the boiling point, and evaporation takes place. The steam is then condensed in the heat exchangers through which the influent water is passing.

Water recovery for this process is usually in excess of 95 percent. Typical applications include high salinity and high scaling water. While the capital cost and operating cost at a low level of salinity would be relatively high; vapor compression evaporation would allow a high percentage of water recovery; be insensitive to organics, and be subject to reduced scaling problems.

Because of the low salinity of the feed water, it was considered that this method would be best utilized in combination with other processes, such as reverse osmosis, for recovery of brine reject streams. In such a system, the increased overall water recovery rate would reduce the required size of upstream treatment processes.

Selection of Overall Demineralization Systems

For the reclamation facility, reverse osmosis was selected for bulk desalination of the brackish water, although it exhibited the lowest water recovery rate. This selection was based on its proven reliability for treating lime softened wastewater effluents, initial capital investment, and operational requirements. Electrodialysis was eliminated because of the potential for organic fouling. At approximately 2,450 mg/l total dissolved solids, the wastewater to be demineralized is too dilute for the costs of distillation processes to be competitive with reverse osmosis. Conversely, to demineralize the wastewater by ion exchange methods alone would be prohibitively expensive in regenerant chemical consumption.

Once reverse osmosis was selected as the method for bulk desalination, a study of overall demineralization schemes was made. The alternatives evaluated were:

1. Reverse osmosis followed by ion exchange: The reverse osmosis process would be capable of providing 200 to 500 mg/l total dissolved solids effluent for the desalting (refinery process) and cooling tower water demands. Boiler feed water would require additional deionization. The requirements for boiler feed water of less than 20 micromhos/cm and less than 0.5 mg/l SiO_2 could reliably be met by a two-bed ion exchange system demineralizing a portion of the permeate from the reverse osmosis process. The reverse osmosis process would reduce the TDS by approximately 90 percent and thus would reduce considerably the chemical regeneration requirements for the downstream ion exchange process. Overall estimated recovery for this system is about 72 percent. While this alternative would have the lowest capital investment cost, the operating costs would be very high because of the expense of regenerant chemicals required by the ion exchangers.

2. Reverse osmosis followed by a second reverse osmosis system and in turn by a two-bed ion exchange system as a polishing unit: While this scheme would have an investment cost considerably higher than the previous scheme, the operational costs would be significantly less. Product water from the primary reverse osmosis train would serve as feed for the cooling tower and refinery desalter water makeup and for feed to the secondary reverse osmosis trains. Whereas recovery in the primary reverse osmosis trains would be limited to 75 percent because of scaling potential, 90 percent water recovery would be possible in the secondary trains. The quality of the product water from the secondary reverse osmosis trains is expected to meet boiler feed water requirements most of the time. The ion exchange units would act as a final quality safeguard, and, because of the high quality feed from the secondary reverse osmosis trains, would require infrequent regeneration. The two reverse osmosis trains, in series, would provide more than adequate protection against biological fouling of the ion exchangers. Reject stream from the secondary reverse osmosis trains could be recycled to the primary trains resulting in an overall water recovery of about 76 percent.

3. Reverse osmosis to produce cooling tower and refinery desalter water makeup and vapor compression evaporation of the reverse osmosis reject to produce boiler feed water: This desalination scheme would have the highest capital investment cost; however, the high recovery, exceeding 95 percent, would allow a reduction in the size of the upstream liquid treatment processes which would balance the higher cost of the demineralization equipment. Operational costs would be significantly higher than that associated with Alternate 2. An important advantage of this scheme is that boiler feed water could be produced independently, in case of emergency, by feeding the effluent from the chemical/physical treatment system directly to the evaporation unit.

Based on the above analyses, Alternative 2 was selected on cost-effectiveness and the increased reliability and simplicity afforded by ion-exchange.

C. WASTEWATER RECLAMATION FACILITY PROCESSES

Chemical Treatment System (See Figure 1)

Equalization:

The WRF will receive up to 20,000 m^3/day of secondary treated wastewater effluent, which will be pumped by means of two constant speed pumps operating on an intermittent basis, over a distance of 19.2 km to two surge ponds located northwest of the refinery. Increased (or reduced) water demands will be met by pumping for a longer (or shorter) period of time as required. Pumping operations will be controlled by a timer or through level controls in the surge ponds.

Chlorine will be injected into the force main at the Riyadh wastewater pumping station in order to inhibit the growth of slime producing bacteria, which could seriously affect pumping efficiencies. The chlorine will also suppress anaerobic sulfate-reducing bacteria whose metabolic action would produce hydrogen sulfide. Provisions will be made for intermediate hypochlorite injection points along the force main route for periodic shock treatment.

An additional chlorine feed point is provided on the force main discharge at the surge ponds for suppression of odors. Chlorination was considered necessary since it is unlikely that a chlorine residual could be maintained throughout the entire length of the force main.

The effluent dissolved oxygen level will be zero after the six hour transmission time to the WRF. The effluent will be accumulated in two aerated, complete-mixed surge ponds. Baffling is provided at the discharge end of each pond to aid in grit separation.

The ponds eliminate septicity by aeration, improve homogenization by keeping organic solids in suspension, and provide flow equalization between the WTP and the WRF. It is expected that the aerated storage will also reduce BOD to or below the mean value given in Table 4.

The ponds each have a capacity of 20,000 m3. They will normally be operated in parallel thereby providing a total of two days' detention, based on the design water requirement. A 6.1 m side water depth will be maintained in the ponds by level controllers for proper operation of the diffused aeration system.

Aeration and mixing of the ponds will be accomplished by means of a static-tube type diffused aeration system fed by rotary positive blowers. One of the reasons for using the submerged static tube aeration system rather than surface mechanical aerators was to help cut down on evaporation losses due to excessive surface agitation. Another reason was the simpler maintenance and access to mechanical equipment that would be provided.

Because of the wastewater characteristics, the potential exists to remove some of the alkalinity from the wastewater effluent in the surge ponds should extensive nitrification occur.

The calcium ion can be removed using lime, if enough calcium alkalinity exists (1.0 mg/l alkalinity removed reduces calcium ion concentration by 0.4 mg/l). Further calcium reduction must be accomplished by adding more alkalinity in the form of soda ash (1.0 mg/l calcium ion reduction requires 2.65 mg/l soda ash). For every mg/l of ammonia-nitrogen oxidized biologically to nitrite or nitrate in the surge ponds, up to 7.2 mg/l of soda ash must be added for calcium ion removal.

Minimization of nitrification so as to keep the requirement for soda ash low, will be achieved by limiting the maximum detention time in the surge ponds to two days. This means that at low flow rates only one pond will be in use. Also, the presence of various organics that are toxic to nitrifying organisms may inhibit or reduce nitrification.

First-Stage Lime Treatment:

Stringent chemical treatment effluent criteria necessitated a closely controlled lime softening clarification process for calcium reduction, suspended solids removal and silica reduction. Pilot plant testing was not possible, and only limited jar test data were available; hence, a large degree of flexibility was designed into the plant. Each design requirement and the resulting process guidelines to meet the design requirement are discussed hereinafter.

Suspended Solids Reduction. Removal of suspended solids will be achieved by chemical coagulation and clarification in the lime softening process. The design of the lime treatment system included the following criteria to facilitate suspended solids removal:

- use of magnesium hydroxide and ferric chloride, as needed for primary coagulants;
- one hour flocculation time in contact with recycled previously precipitated solids;
- one gpm per square foot clarifier hydraulic loading (considered conservative for the very warm waters being treated).

Data from existing wastewater reclamation plants in the U.S.A. indicate that overall suspended solids reduction to less than 10 mg/l can be expected by chemical coagulation and sedimentation.

Silica Reduction. If the reverse osmosis process is to be operated at a recovery rate of 75 percent, there will be a 4 to 1 concentration of silica in the reject stream. Silica reduction is not required as long as the influent silica does not exceed 34 mg/l as SiO_2, which is the maximum observed value. However, designing the chemical treatment system for silica reduction to allow for possible higher future influent silica concentrations above these levels is recommended.

Silica will be removed by adsorption on magnesium hydroxide floc. The amount of magnesium hydroxide required to remove silica depends on silica concentration, pH, temperature, contact time and sludge recirculation. A design value of 4 parts magnesium hydroxide per 1 part of silica to be removed was chosen for up to 50 percent silica reduction. This choice was based on a pH of about 11, a flocculation time of 60 minutes, a temperature of 35°C, and the fact that sludge is to be recirculated.

Increasing the amount of magnesium hydroxide precipitated to the delivered 4 : 1 silica ratio involves raising the pH to 11 by adding lime. However, increasing the lime feed to raise the pH may raise the calcium ion above the allowable concentration. In this event, it will be necessary to add soda ash to reduce the calcium to an acceptable level.

Calcium Reduction. The reverse osmosis manufacturers have recommended a maximum concentration of about 105 mg/l of calcium in the feed water to the reverse osmosis units. This is based on the theoretical solubility of calcium sulfate in clean water at 32.2°C and with the sulfate ion in the feed water at about 1,000 mg/l. Calcium concentrations in the Riyadh WTP effluent range from 228 mg/l to 268 mg/l, with a mean value of 242 mg/l. Reduction of the calcium ion concentration will also decrease the hardness by 20 percent, thereby satisfying the utility water quality criteria. The magnesium hydroxide, precipitated at high pH for silica removal, will also act as a coagulant to enhance colloidal particulate removal. Soda ash will be used to increase the carbonate alkalinity when the influent alkalinity is less than that needed to reduce the calcium to 105 mg/l or less. Facilities will also be provided to utilize carbonated caustic soda, when soda ash is not available.

Most existing wastewater reclamation plants in the U.S.A., utilizing high lime treatment, have had difficulty reducing the calcium ion concentration below influent levels. The reasons for the failure of calcium carbonate to precipitate to the theoretical extent in these plants has been shown to be due to inhibition of carbonate precipitation by various organics and polyphosphates present in wastewater. Limited jar tests performed by Camp Dresser & McKee in Riyadh WTP effluent were inconclusive. However, in anticipation of this potential problem, several precautions have been built into the chemical treatment system to provide the treatment flexibility to overcome this sequestering action. These include:

- provision for adding up to 400 mg/l soda ash to increase the concentration of carbonate ions for precipitation of calcium ions associated with noncarbonate hardness;

- provision for a complete, two-stage lime-soda ash treatment system to maximize the treatment flexibility, and to make possible water softening in both stages;

- recirculation of sludge to facilitate coagulation and reducing the tendency to supersaturation;

- use of magnesium hydroxide and ferric chloride as primary coagulants;

- provision to recover magnesium as magnesium carbonate from the sludge for use as a coagulant;

- provision for added polymer at any of the three points in the flocculation basins as a coagulant aid;

- capacities in both the lime and caustic soda feed systems sufficient to raise the pH to 11.5 or higher, if necessary, to enhance calcium ion reduction and allowing removal of all the magnesium;

- a clarifier hydraulic loading of one gpm per square foot to assure proper solids settling.

Two mechanical mixers in the rapid mix basin blend wastewater infleunt and lime. Caustic soda, soda ash, recycled magnesium carbonate, and recirculated first-stage lime sludge are added as required. A two compartment basin is provided to minimize short circuiting. The detention time in the basin is about 3 minutes at design flow.

Lime addition is controlled by a signal from a pH analyzer mounted at the effluent end of the rapid mix basin. This signal can also be used to control caustic soda feed, if the lime system is not operating. Normally, the dosages of caustic soda and soda ash, when required, are locally set and the amount of each chemical added is influent flow paced.

The amount of recirculated first-stage lime sludge is controlled by a bleed off valve on the discharge of the first-stage lime sludge pumps. The amount of magnesium carbonate recycled to the basin is regulated by a constant level control system in the magnesium carbonate holding tank.

The chemically treated water flows from the rapid mix basin to a distribution channel where the flow is divided into four parallel, rectangular flocculation basins, each with three square compartments in series. Total detention time is 60 minutes at design flow. Each compartment is equipped with an impeller type flocculator. The mixers are variable speed to allow independent control of mixing energy in each compartment. Polymer can be added in any of the three compartments of each flocculation basin.

Water flows from the bottom of the flocculation basins and enters two parallel, rectangular clarifiers, each equipped with three circular sludge collector mechanisms installed in a square well configuration. Withdrawal of sludge by gravity from each sludge collection well is controlled by a manual, air activated butterfly valve located near the sludge pumping station.

Effluent collection troughs are located in the downstream half of each clarifier. The clarified water is then evenly distributed to two first-stage recarbonation basins.

The recarbonation basins are designed so as to accomplish two functions The first is to provide recarbonation of clarified effluent to lower the pH to approximately 9.5, the minimum calcium carbonate solubility level The second is to mix caustic soda, soda ash, or ferric chloride as needed.

Chemical blending is accomplished in a rapid mix compartment at the effluent end of the basin. Chemical addition provides the flexibility in the second-stage basins, so as to increase the carbonate alkalinity with soda ash for removing the calcium ion.

Recarbonation and pH control are to be accomplished in the second compartment of the basin. The quantity of carbon dioxide will be regulated by a signal from a pH analyzer located at the effluent end of the basin.

Second-Stage Treatment:

The first-stage recarbonation basins discharge directly into four second-stage flocculation basins. These basins are identical with respect to the first-stage flocculation basins. Provisions are again made for adding polymer to any of the three compartments of each flocculation basin.

The second-stage flocculation basins discharge into the second-stage clarifiers. The two rectangular clarifiers are also identical to the first-stage clarifiers. The collection trough discharges flow into two second-stage recarbonation basins. Chlorine can be added to the clarifier influent to minimize micro-organism growth in the clarifiers.

The two second-stage recarbonation basins are also designed for two purposes. The first is to convert carbonate alkalinity to bicarbonate alkalinity using carbon dioxide. The pH will be lowered from about 9.5 to around 8.5. The second is to reduce the alkalinity to stabilize the water and prevent the precipitation of calcium carbonate in the utility water storage tanks and treatment processes downstream of the chemical treatment system.

Recarbonation is accomplished in the first two of the basin's compartments. A pH analyzer located at the effluent end of the basin controls the amount of carbon dioxide added.

Sulfuric acid is added in a rapid mix zone at the end of the second compartment. The third compartment provides adequate detention time to allow reduction of the alkalinity prior to discharge. The dosage of acid is controlled by a flow signal from the influent flowmeter. The amount of acid used is manually established based on the degree of excess alkalinity in the process.

The recarbonation basin effluent flows into the utility water pumping station wet well from which it can be pumped to the utility water storage tanks, cooling towers or discharged directly to the gravity filters. Chlorination of the stored utility water will be provided to assure a disinfected water supply. The average quality characteristics of this water are shown in Table 8.

Physical Treatment System (See Figure 2)

Cooling Tower:

After storage in the utility water storage tanks, the chemically treated and chlorinated wastewater is to be fed into a cooling tower to reduce its temperature from about 45^oC to 25^oC. This temperature reduction is necessary to minimize reverse osmosis membrane degradation and compaction. Provisions have been made for pumping second-stage recarbonation effluent directly to the cooling towers if needed.

The cooling tower will be of the two-cell induced draught type. Each cell will be rated at a design flow of 971.4 m^3/hr. Sulfuric acid will be added to the stored influent water in order to adjust the pH from 8.5 to 6.0 to reduce the potential for calcium carbonate scaling and to prevent delignification and consequent weakening of wood packing at elevated pH values. Likewise, provisions will be made for chlorine addition to minimize slime growth on the packing.

Gravity Filtration System:

The wastewater will flow from the cooling tower to a chemical mix basin where a polymer will be added to assist in any removal of colloidal or particulate solids. Periodic chlorination of the filter bed will be done for algae growth control.

The flow will then enter an influent box and be split equally between four dual-media (coal-sand) gravity filters. The filters will operate so that the rate of flow through the filters will automatically respond to an increase or decrease in downstream process water demand.

The influent design flow to the gravity filters is 942.3 m^3/hr with a surface loading rate of 3 gpm/sf.

Backwash of filters will be initiated by high filtered water turbidity, excess headloss or by timer, and will be automatic. Filters will be equipped for air-water backwash. The temperature of the backwash water will be monitored and used to automatically modify preset wash rates.

Under design conditions, an effluent turibidity not greater than 0.5 TU (turbidity units) will be achieved.

Carbon Adsorption System:

There is some operating experience in the U.S.A., such as the Pomona water reclamation plant in Los Angeles, California, which indicates that activated carbon pretreatment for organics removal is not necessary for successful reverse osmosis operation.

There are, however, several reasons for providing activated carbon in this WRF process scheme. To begin, the available data from the Riyadh WTP reveals that the effluent COD values average approximately twice the COD values of the secondary wastewater used in the Pomona studies. In addition to this, the wastewater characteristics are uncertain due to the variations in available data and wastewater treatment plant expansion. Carbon adsorption will effectively reduce the organic loading on the reverse osmosis units. This, in turn, substantially reduces the cleaning down-time and prolongs the life of the reverse osmosis membranes.

The wastewater will undergo carbon adsorption treatment following gravity filtration. Eight pressure vessels will contain the activated carbon, with two additional columns acting as both regenerated carbon storage and standby adsorption columns. Two sets of four adsorption columns each will comprise the system; one set will operate as upflow adsorption columns and the other set will operate as downflow adsorption columns. The two sets of columns will operate in a series mode with the downflow set following the upflow set. Each of the four columns in each set will operate in parallel. The intent of utilizing both upflow and downflow adsorption operation modes is to produce the maximum degree of adsorption while at the same time utilizing the inherent filtration capability of downflow columns to prevent activated carbon fines from being carried over to the reverse osmosis units in the carbon adsorption process effluent.

The influent design flow to the carbon adsorption system is 673.2 m^3/hr with an average COD concentration of approximately 100 mg/l. The volume of carbon required is based on a total retention time within the carbon and on an empty bed basis of 30 minutes. The surface loading of the carbon columns is 6.5 gpm/sf, which is in the low end of the acceptable 5 to 10 gpm/sf design range.

The effluent after filtration and carbon adsorption will have a COD of 10-20 mg/l, and a turbidity of 0.5 TU.

Spent carbon from the adsorption columns will be transferred from the adsorption columns to these spent carbon storage tanks prior to being transferred to the dewatering bins on top of the regeneration furnace. The two spent carbon storage tanks each have the capacity for one adsorption column carbon bed volume.

Carbon Regeneration System:

Two dewatering bins with two dewatering screw conveyors are located on top of the carbon regeneration furnace. The bins are designed with total capacity for one day of spent carbon feed. The dewatering screw conveyors are utilized to convey the spent carbon into the furnace, as well as to assure adequate dewatering of the spent carbon to about 50 percent moisture content.

Reactivation of the spent carbon is accomplished by thermal regeneration. A multiple hearth furnace will volatilize and oxidize the absorbate within a controlled atmosphere at a temperature of between 815 and 930°C. The hot regenerated carbon will then be cooled in a quench tank upon leaving the furnace prior to transfer to storage.

The furnace size design is based on the carbon consumption rate, furnace loading, provision for future carbon consumption and potential emergency conditions. Carbon consumption was estimated to be two pounds per pound of COD adsorbed. At design flow with 90 percent removal of 100 mg/l influent COD, the furnace loaidng is 54 lbs/day/sf.

Demineralization System (See Figure 3)

Hollow fiber and spiral wound membranes are the two most commonly installed reverse osmosis membrane types. Both configurations use membranes from either the cellulose (tri)acetate or polyamide families, each with specific properties, advantages, and disadvantages. Hollow fiber modules require the highest degree of pretreatment of all membrane types, and have a maximum recommended continuous operating temperature of 35°C for polyamide fibers and 30°C for cellulose triacetate fibers. The spiral wound membrane configuration results in elements that are less prone to fouling than the hollow fiber elements. The maximum recommended continuous operating temperature for cellulose acetate spiral wound elements is 30°C, and for the polyamide elements 40°C.

Spiral wound membranes were recommended for use at this WRF because of greater resistance to fouling. These units have been extensively pilot tested on treated wastewater and about a half dozen full scale installations have been operated with treated wastewater as a supply. Conversely, while hollow fiber units have been widely used on salt and brackish water supplies, there are no full scale installations of hollow fiber units utilizing treated wastewater as a direct supply.

Polyamide was selected over cellulose acetate as a membrane material because of polyamide's increased water permeability rate, increased chemical stability, increased allowable pH range (3-12 rather than 4-7) which permits the use of more concentrated cleansing agents, increased resistance to biological attack, increased temperature range, and increased resistance to compaction resulting in reduced flux decline. The major disadvantage of a polyamide membrane is that it cannot tolerate residual chlorine as can cellulose acetate. However, the aforementioned advantages greatly outweighed this disadvantage. Also, polyamide's increased resistance to biological degradation and the increase in allowable types and strengths of cleaning solutions eliminates any requirement for chlorination of the feed water.

Reverse Osmosis Pretreatment System:

After carbon filtration, the treated water will enter a dechlorination basin to remove all residual chlorine with either sodium bisulfite or sodium sulfite used as the dechlorination chemical. From the dechlorination basin, the water will flow to the cartridge filter feed wet well. Water from the dechlorination basin will be combined with the brine reject from the secondary reverse osmosis system for a total design feed volume of 16,630 m^3/day. Two centrifugal pumps, one as standby, will pump this flow through the cartridge filters and to the primary reverse osmosis feed wet well.

Sulfuric acid for pH control and sodium hexametaphosphate (SHMP) for scale inhibition will be added to the cartridge filter feed piping. pH will be maintained at about 5.7 which will reduce the likelihood of calcium carbonate scaling and is within the pH range at which monovalent cations are most efficiently rejected by the polyamide membrane. While the 75 percent recovery of the primary reverse osmosis system was selected to maintain the waste brine concentrations below the theoretical calcium sulfate precipitation point (saturation would occur at 80 percent recovery), the addition of SHMP will provide a factor of safety for concentration gradients within the membrane elements of transient increases of ions in the feed water.

Four cartridge filters will be supplied (one as standby) to serve as a final screen prior to reverse osmosis and as a mixing device for the upstream chemical addition. Filter cartridges will be of polypropylene with stainless steel internals.

Primary Reverse Osmosis System:

For the primary reverse osmosis system, five assemblies of trains will be supplied. Each train will be rated at a feed flow of 173.3 m^3/hr with a water production rate of 129.9 m^3/hr at 75 percent recovery. Four trains will be sufficient to treat the design flow of 16,630 m^3/day. At this flow, with a TDS concentration of 2,600 mg/l, a total quantity of about 12,470 m^3/day will be produced with an anticipated TDS concentration of less than 200 mg/l, which is well below the required TDS of 500 mg/l for cooling tower and refinery desalter makeup. The brine reject will total 4,160 m^3/day with TDS of about 9,800 mg/l. This reject stream will be sent to evaporation ponds.

Each primary reverse osmosis train will consist of a two-stage arrangement of pressure tubes with the reject from the first-stage tubes serving as the feed to the second-stage pressure tubes. Each stage will provide 50 percent water recovery for an overall recovery of 75 percent. Product water discharge pressure will be sufficient to feed either existing water storage tanks or the secondary reverse osmosis feed wet well, or both, simultaneously. Each primary reverse osmosis train will be mated to one vertical turbine feed pump rather than having larger pumps feeding multiple trains. This system was selected since it provides easier operational control than a multiple arrangement. One pump will be manifolded to assure a standby for the other four pumps.

In addition to the above, the primary reverse osmosis trains will be able, at times when water requirements are less than the design flow, to reduce recovery to 70 percent. This will permit a higher allowable calcium concentration in the feed water and significantly reduce chemical requirements in the lime softening section of the water reclamation plant. The average quality characteristics of this water are shown in Table 9.

Secondary Reverse Osmosis System:

The secondary reverse osmosis system will consist of three trains. Each train will consist of three stages of pressure vessels for an overall recovery of 90 percent. Again, each assembly will be mated to a vertical turbine feed pump, with one pump manifolded as a standby pump. Each train will be rated at a feed flow of 2,380 m^3/day of primary reverse osmosis product water, a production rate of 2,140 m^3/day with an anticipated TDS of less than 40 mg/l. The average quality characteristics of this water are shown in Table 10.

The reject stream of 240 m^3/day will have a TDS of about 1,600 mg/l and will be suitable for blending with the feed to the primary reverse osmosis system.

A chemical cleaning system and automatic flushing system complete the reverse osmosis installation. The flushing system storage tanks provide sufficient capacity for emergency flushing of all eight trains.

Ion Exchange System:

Product water from the secondary reverse osmosis trains is fed to forced draft air decarbonators. After decarbonation, the water will be pumped through the ion exchange system which consists of two parallel trains of two bed ion exchange columns. Each train will be sized for 100 percent of the design flow. Product water from the ion exchange columns will have a specific conductivity of less than 20 micromhos. Total usable production will average 4,170 m^3/day. The average quality characteristics of this water are shown in Table 11.

At times, the quality of the secondary reverse osmosis system is expected to meet the required boiler feed water quality. Provisions are made for bypassing the ion exchange system under these conditions to save regeneration chemicals. Also, it will be possible to provide split treatment by bypassing a portion of the flow which will also result in saving of regeneration chemicals.

Lime and Magnesium Reclamation System

Lime sludges are produced in the chemical treatment section from the coagulation/softening of the wastewater. The lime reclamation system collects, concentrates, and dewaters these sludges so that lime can be recovered by recalcining. The volume of residual waste materials requiring disposal is thus minimized.

Sludge Handling System:

Carbonation of the sludge dissolves most of the magnesium hydroxide present, creating a soluble magnesium carbonate solution. The solution can be separated from the sludge by gravity thickening and recycled directly to the first-stage lime treatment system to augment the magnesium present, so as to precipitate more magnesium hydroxide and thus increase the silica removal and assist in coagulation of the suspended and precipitated solids.

Removing magnesium from the calcium carbonate sludge also increases the purity of the recalcined lime and improves the dewaterability of the sludge, thereby decreasing the required size of the dewatering and recalcination systems.

Dissolving magnesium hydroxide in the sludge can be accomplished by diffusing carbon dioxide in it for a period of time. Carbon dioxide will selectively dissolve the magnesium hydroxide at a pH between 8.3 and 8.5. If the pH decreases below 8.3, the calcium carbonate will be converted slowly to soluble calcium bicarbonate, which, if recycled, will increase the calcium hardness in the chemical treatment process. Therefore, pH control is provided by an analyzer located on the effluent end of the sludge carbonation basin to control the amount of carbon dioxide added.

A two compartment basin to prevent short circuiting and a 60 minute detention time is provided. A mixer will be installed in each compartment to promote carbon dioxide diffusion, facilitate the dissolving reaction, and keep the sludge solids in suspension.

From the sludge carbonation basin, the solids are thickened in two gravity thickeners to 10-30 percent. Second-stage lime sludge, containing practically all calcium carbonate, will be fed directly to the thickeners. Any portion of the supernatant that has the dissolved magnesium carbonate that is not recycled will be wasted to the evaporation pond.

The thickener underflow, with the sludge solids containing approximately 85 percent calcium carbonate, will be pumped to either the filter press system for dewatering and further processing for reclamation, or to two waste sludge ponds.

Filter Press System:

The filter press system consists of two plate and frame filter presses. The thickened calcium carbonate sludge will be dewatered to 50-60 percent dry solids. Under design specifications, the filter press system will be able to process 18,970 kg/day of dry solids over a period of 18 hours. Each filter press will be rated at 100 percent of design conditions.

Thickened sludge will be fed by progressive cavity pumps to a sludge holding tank. A slow speed mixer will be provided to maintain solids homogenization. Sludge is fed to the press by two sets of pumps. First by low head centrifugal pumps for initial filling and then by high pressure (225 psi) positive displacement pumps for filtration. The filtration cycle time, which is dependent upon the percentage of solids feed, is expected to average one hour which includes turnaround time for cake discharge. The filtrate from the sludge dewatering operation will flow to the filtrate tank for possible recovery with the thickener supernatant.

The dewatered sludge cake will be stored below each filter press in storage bunkers and conveyed by a drag flight, chain-type conveyor to the recalcination system. Provisions will be made for intermediate discharge to a truck loading station.

Lime Recalcination System:

The dewatered sludge will be fed to a multiple hearth furnace designed to recalcinate 18,970 kg/day of dry solids. For every 100 kg of sludge (dry solids basis) recalcinated, 45 kg of reactive CaO is expected. The lime produced is estimated to be 70 percent CaO or higher. Exhaust gases from the calciner will be recovered for use in the recarbonation basins and the sludge carbonation basin. It is estimated that it should be possible to produce by recalcination all of the CaO and CO_2 required in the treatment process.

Lime Feed System:

The recalcinated lime will be pneumatically conveyed to two lime storage bins, which provide 30 days' storage. Stored lime will then be pneumatically transferred to two lime use bins located above two volumetric belt type lime feeders. Each feeder will be controlled by a pH signal from the rapid mix basin. The feeders discharge to two lime paste slaking systems (one as standby), each capable of slaking quicklime at a maximum rate of 10,600 kg/day.

D. COSTS

Accurate cost figures are not available at present, as the WRF equipment is, as of this date, being bid. The total installed cost of this facility is estimated to be between $40-50 million. The cost of produced water is difficult to estimate as there will be three different levels of quality. Conservatively, it is estimated that the operation and maintenance costs of the demineralized water will fall in the range of 80 to 90 cents per thousand gallons. The above costs in Saudi Arabia will be significantly greater.

TABLE 1

UTILITY WATER FOR HOSE STATION AND FIRE WATER

Maximum Flow	100 gpm (22.7 m^3/h)
Normal Flow	50 gpm (11.4 m^3/h)
BOD_5	85% reduction
SS	95% reduction
TDS	No substantial increase over WRF influent
Hardness	20% reduction
SiO_2	40% reduction
Stable and Disinfected	

TABLE 2

COOLING TOWER MAKEUP AND DESALTING (REFINERY PROCESS) WATER

Maximum Flow	1,410 gpm (320.5 m^3/h)
Normal Flow	950 gpm (215.9 m^3/h)
SS	1 mg/l (maximum)
TDS	500 mg/l (maximum)

TABLE 3

BOILER FEED WATER

Maximum Flow	765 gpm (173.9 m^3/h)
Normal Flow	500 gpm (113.6 m^3/h)
SS	1 mg/l (maximum)
SiO_2	0.5 mg/l (maximum)
Specific Conductance	20 micromhos/cm (maximum)

TABLE 4

SUMMARY OF AVAILABLE DATA
RIYADH WASTEWATER TREATMENT PLANT

EFFLUENT DATA

Constituent	Minimum	Maximum	Mean	Number of Samples	Statistical Analysis 10%	Median	90%
pH	7.2	8.1	7.6	97			
Alk mg/l ($CaCO_3$)	370	600	442	129	405	442	480
TH mg/l ($CaCO_3$)	1,000	1,060	1,025	4			
Ca mg/l	228	268	242	4			
Mg mg/l	93	103	97	4			
TDS mg/l	2,120	2,730	2,330	129	2,210	2,330	2,450
BOD mg/l	7	93	43	123	24	43	62
COD mg/l	108	316	223	129	169	222	275
SS mg/l	45	137	73	129	51	73	95
NH-$_3$N mg/l	16	58	34	123	28	34	41
NO_3 mg/l	35.2	110	72.6	2			
SiO_2 mg/l	23.2	33.7	29.6	5			
Cl mg/l	350	650	489	84	392	488	585
SO_4 mg/l	615	947	778	4			
PO_4 mg/l	11	12	11	3			
Na mg/l	836	993	915	2			
Turbidity (TU)	10	20	15	2			

---

[a] Cl data taken from Riyadh STP operating records, December 1975 to July 1977.

[b] pH, Alkalinity, BOD, COD, SS, NH_3-N. TDS, NO_3 data taken from Riyadh STP operating records, January 1978 to May 1978.

[c] All other data taken from miscellaneous individual analyses.

TABLE 5

INFLUENT WASTEWATER CHARACTERISTICS

Flow Rate, m^3/d	20,000
pH	7.0 to 8.0
Temperature, °F	110 maximum
Dissolved Oxygen, mg/l	0
Ammonia, mg/l as Nitrogen	55 maximum
SiO_2, mg/l	35 maximum
Suspended Solids, mg/l	194 maximum
Biochemical Oxygen Demand, mg/l	225 maximum
Chemical Oxygen Demand, mg/l	275 maximum
Total Dissolved Solids, mg/l	3,300 maximum
Total Hardness, mg/l as $CaCO_3$	1,060 maximum
Total Alkalinity, mg/l as $CaCO_3$	230 minimum
Calcium, mg/l	268 maximum
Magnesium, mg/l	103 maximum
Sodium, mg/l	995 maximum
Iron, mg/l	2 maximum
Sulfate, mg/l	1,000 maximum
Chloride, mg/l	650
Phosphate, mg/l	15

TABLE 6

PRETREATMENT QUALITY GOALS

Parameter	Goal
BOD, COD	95% reduction
Temperature	25°C
Suspended Solids	99% reduction
Turbidity	0.5 TU (turbidity units)
Silica	50% reduction of SiO_2
TDS	No substantial increase over WRF influent
Hardness	Reduction to the extent possible in the coagulation/softening/sedimentation process

The fire and hose station supply must be chemically and biologically stable and disinfected.

TABLE 7

SUMMARY OF UNIT PROCESSES EVALUATED

Pretreatment

- Grit Removal
- Coagulation/Softening/Sedimentation
- pH Adjustment
- Cooling Towers
- Filtration
- Activated Carbon Adsorption

Demineralizotion

- Electrodialysis
- Reverse Osmosis
- Ion Exchange
- Vapor Compression Evaporation

TABLE 8

AVERAGE QUALITY CHARACTERISTICS OF CHEMICALLY TREATED WATER

pH	Stable per Langelier Index
Ammonia, mg/l as Nitrogen	45 - 50
SiO_2, mg/l	15 - 20
Suspended Solids, mg/l	10
Chemical Oxygen Demand, mg/l	100 - 500
Total Dissolved Solids, mg/l	Somewhat less than influent (2500)
Total Hardness, mg/l as $CaCO_3$	200 - 300
Total Alkalinity, ma/l as $CaCO_3$	40
Calcium, mg/l	105

TABLE 9

AVERAGE WATER QUALITY FROM PRIMARY REVERSE OSMOSIS SYSTEM

pH	5
Ammonia, mg/l as Nitrogen	7 - 10
SiO_2, mg/l	5
Chemical Oxygen Demand, mg/l	3
Total Dissolved Solids, mg/l	200 - 300
Total Hardness, mg/l as $CaCO_3$	50
Total Alkalinity, mg/l as $CaCO_3$	5
Sulfate, mg/l	45 - 60

TABLE 10

AVERAGE WATER QUALITY FROM SECONDARY REVERSE OSMOSIS SYSTEM

Ammonia, mg/l as Nitrogen	1 - 3
SiO_2, mg/l	0.75 - 1.5
Chemical Oxygen Demand, mg/l	1
Total Dissolved Oxygen, mg/l	15 - 25
Total Hardness, mg/l as $CaCO_3$	3 - 5

TABLE 11

AVERAGE WATER QUALITY FROM ION EXCHANGE SYSTEM

Ammonia, mg/l as Nitrogen	0
SiO_2, mg/l	0.5
Chemical Oxygen Demand, mg/l	0
Total Dissolved Solids, mg/l	10

Figure 1

CHEMICAL TREATMENT SYSTEM

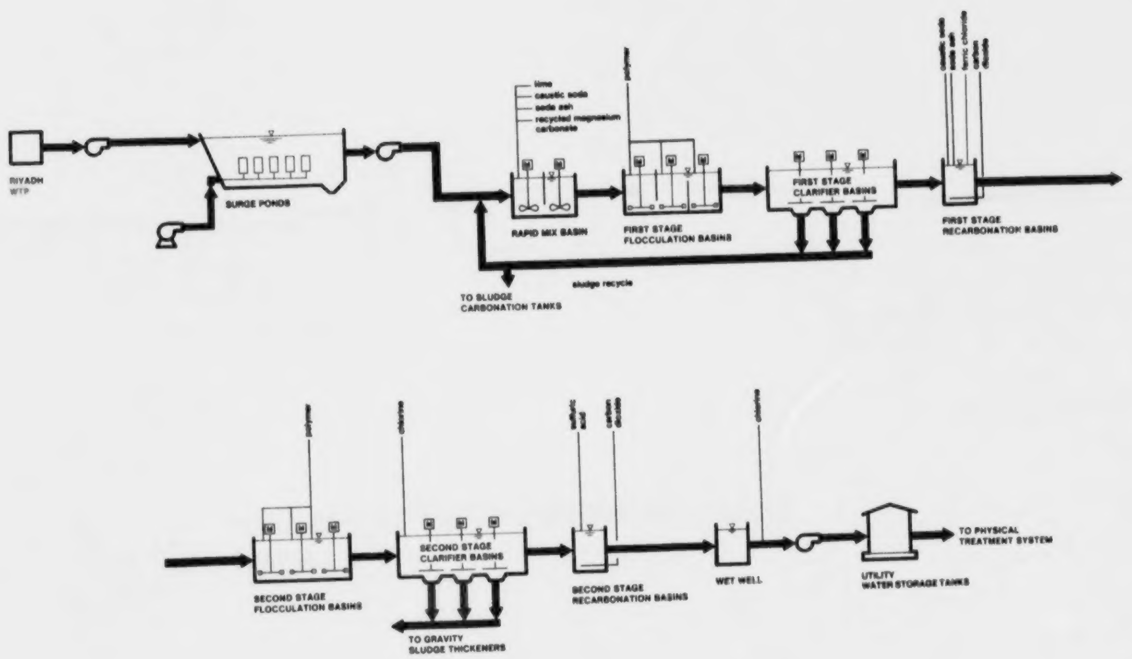

RIYADH WTP
SURGE PONDS
lime
caustic soda
soda ash
recycled magnesium carbonate
RAPID MIX BASIN
polymer
FIRST STAGE FLOCCULATION BASINS
FIRST STAGE CLARIFIER BASINS
caustic soda
soda ash
ferric chloride
carbon dioxide
FIRST STAGE RECARBONATION BASINS
sludge recycle
TO SLUDGE CARBONATION TANKS
polymer
SECOND STAGE FLOCCULATION BASINS
chlorine
SECOND STAGE CLARIFIER BASINS
TO GRAVITY SLUDGE THICKENERS
sulfuric acid
carbon dioxide
SECOND STAGE RECARBONATION BASINS
chlorine
WET WELL
UTILITY WATER STORAGE TANKS
TO PHYSICAL TREATMENT SYSTEM

Figure 2
PHYSICAL TREATMENT SYSTEM

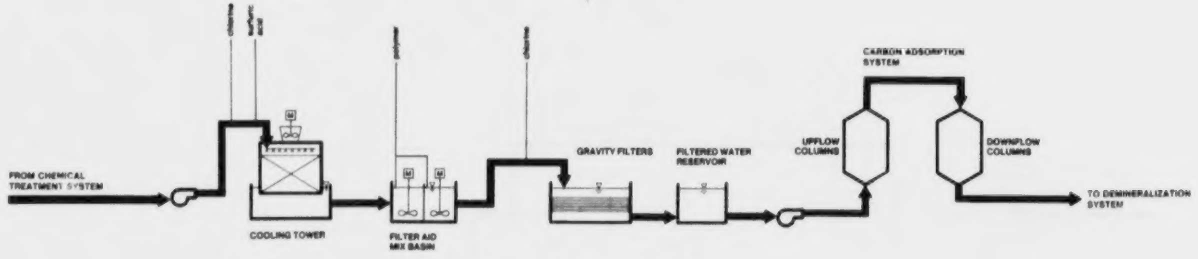


Figure 3

DEMINERALIZATION SYSTEM

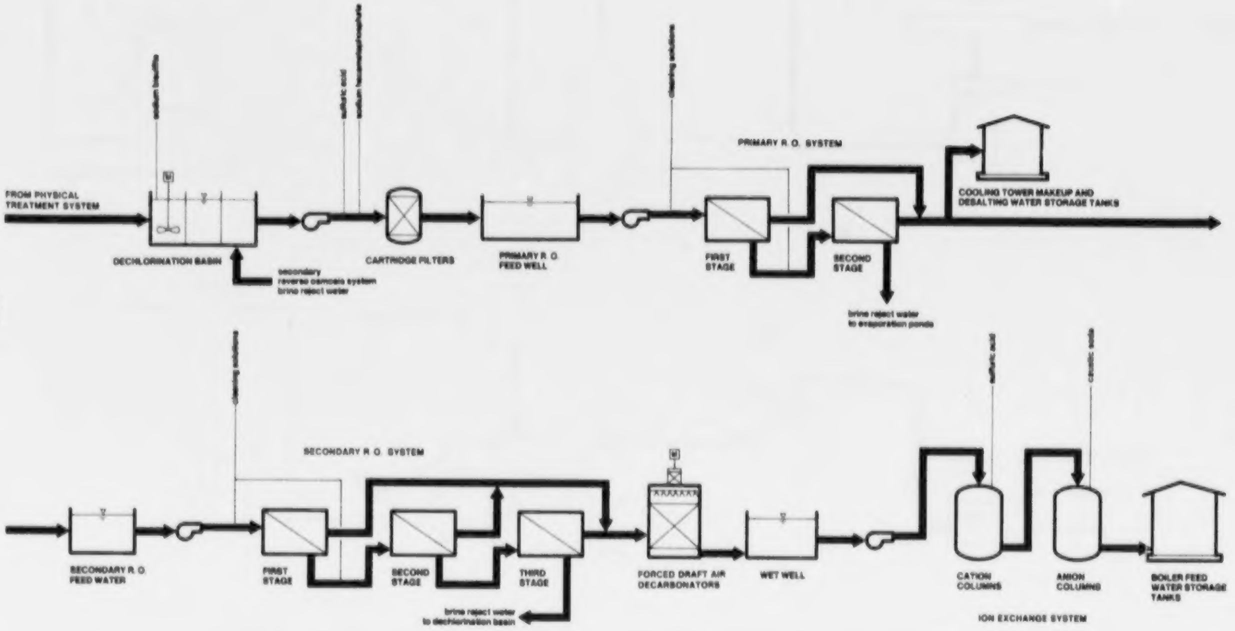
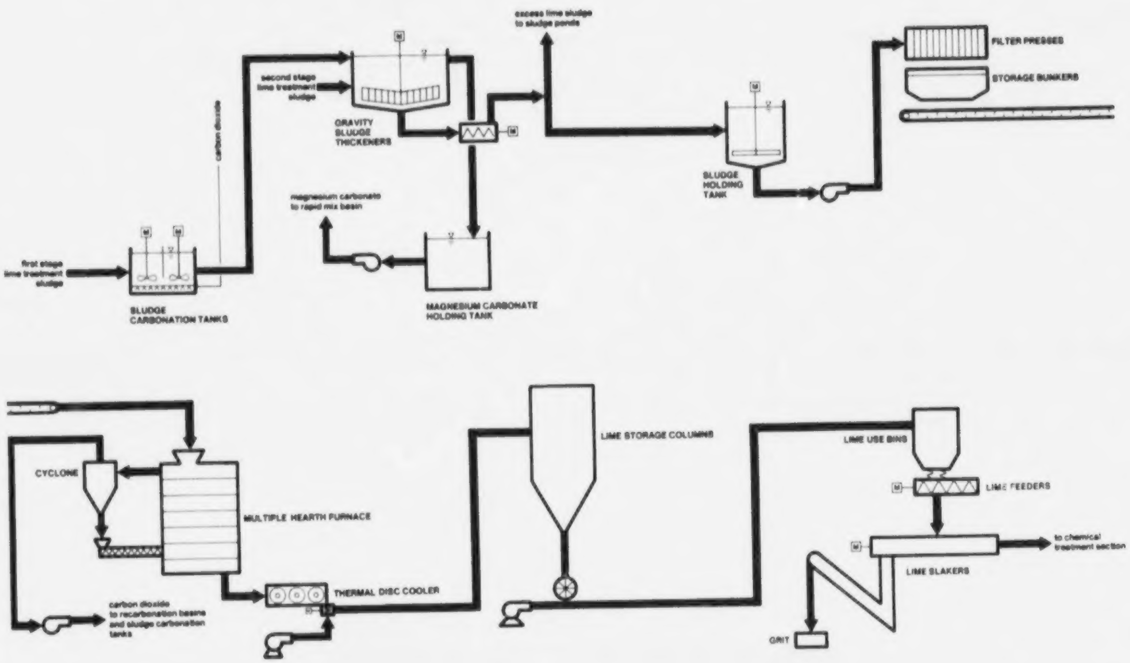


Figure 4

LIME AND MAGNESIUM RECLAMATION SYSTEM



ECOLOGICALLY BALANCED WASTEWATER RENOVATION SYSTEM

Amit Basu
Environmental Engineer
Engineers India Limited
New Delhi
INDIA

Wastes produced as a result of human activities are recognised today as the 'Resources Out of Place'. Community wastewater is one such throw away resource in which all the nutrients extracted from soil by crops are present along with water. Utilisation of these waterborne nutrients for aquaculture and agriculture has achieved tremendous importance in present times in view of the fact that resource conserving urbanisation has become an integral part of our urban survival and healthy development. Ecologically balanced wastewater renovation system proposed for handling the combined sanitary and treated oily wastes generated within the copper ore handling complex at Malanjkhand in the State of Madhya Pradesh, India, is an outcome of the above concept. System design and management are based on the initial findings of the Department of Science and Technology, Government of India, sponsored research project conducted by the author at the Environmental Engineering Department, Jadavpur University, Calcutta.

SYSTEM CONCEPT

Ecologically balanced wastewater renovation system functions utilising the solar energy, employs natural recycling processes for the renovation of wastewater and helps maintaining the 'Biogeochemical Cycles'(1). Conceptually, the proposed system comprises a multiple celled stabilisation pond, as the central wastewater treatment unit surrounded by a green belt developed by applying the treated effluent as the irrigant and digested sludge as mannure. The ponds stabilise the organics present in the wastewater through algae-bacterial symbiosis in upper layers and through anaerobiosis at the pond bottom. The main purpose of treatment is to make the wastewater sufficiently innocuous for subsequent uses. The landscape developed around the pond is available for multiple use-benefits, namely

- agri-horticulture,
- nutrient conservation,
- recreation, and
- wild life preservation.

A schematic representation of the system concept is given in Figure 1.

RESEARCH AND STUDIES

In order to transfer the concept to the field, studies were initiated towards -

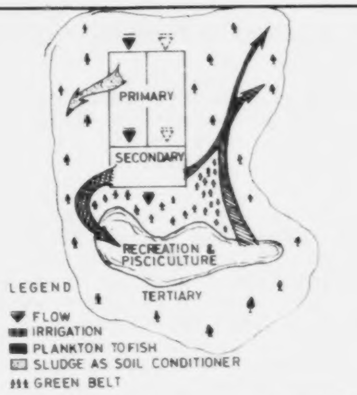


FIG. 1 : SYSTEM CONCEPT - A SCHEMATIC REPRESENTATION

- evaluating the agricultural use potential of community wastewater and its digested sludge.
- delineating the planning and management details of pisciculture in treated community wastewater.
- determining the treated wastewater application rate on land for different soil-vegetation complexes with and without application of sludge.

Agricultural Use Potential

Use of community wastewater for agricultural irrigation purposes in a particular situation can be recommended only when the complete physical and chemical characteristics of the wastewater are known. This is essential for selection of the type of crop to be raised, detection of the presence of any phytotoxic element and effect suitable remedial measures against the failure of crops. The analysis also helps to estimate the quantity of fertilising ingredients such as nitrogen, phosphorus, potassium available in the wastewater and the amount to be supplemented to get a desired crop yield. Ganguli Bagan Housing Estate located about three kilometers south of Jadavpur University was considered a suitable place for the collection of treated wastewater and digested sludge samples for the study. The Housing Estate receives water from ground source, per capita water supply being approximately 400 l/day. Wastewater generated is entirely domestic in nature and is treated in a biological filter with primary and secondary clarifiers alongwith sludge lagoons within the housing complex. Samples were analysed for various physical and chemical parameters which have relevance to the agricultural use. The summarised analysis results of treated wastewater and digested sludge are presented in Table 1 and Table 2, respectively.

TABLE 1

Characteristics of Treated Wastewater : Ganguli Bagan Biofiltration Plant

Constituent	Range		Average
	Min.	Max.	
pH	7.50	8.85	8.40
Electrical Conductivity*	1.32	4.42	3.11
Total Dissolved Solids	2060	3588	3046
Total Suspended Solids	25	40	30
BOD_5 (20°C)	8.30	68.00	18.00
Ammonia-N	2.22	10.04	5.04
Kjeldahl-N	2.80	12.40	7.33
Nitrite-N	Trace	Trace	Trace
Nitrate-N	1.34	4.80	2.30
Total Phosphorus	0.96	2.25	1.50
Filtrable Phosphorus	0.40	1.80	1.18
Potassium	11.30	16.50	12.08
Chloride	709	1645	1010
Sulphate	16	140	54
Sodium	310	900	543
Iron**	100	1375	617
Manganese**	30	440	254
Zinc**	25	200	79
Boron**	109	187	140

*Expressed as millimhos/cm. at 25°C.

**Expressed as µg/l.

Except pH, all values expressed as mg/l.

TABLE 2

Characteristics of Sludge : Ganguli Bagan Biofiltration Plant Sludge Lagoon.

Constituent	Range		Average
	Min.	Max.	
pH	6.60	7.40	6.97
Organic-carbon	7.52	8.38	7.85
Nitrogen	1.06	1.33	1.08
Phosphorus (P_2O_5)	1.12	1.43	1.41
Pottasium (K_2O)	0.74	1.16	0.94
Calcium	6.13	8.03	7.12
Magnesium	0.33	0.77	0.56
Sodium	0.50	0.78	0.57
Iron	3.45	4.38	4.07
Copper	0.03	0.05	0.04
Lead	0.03	0.15	0.08
Manganese	0.05	0.06	0.06
Zinc	0.16	0.53	0.39

Except pH, all values are reported in percentage by weight on dry sludge basis.

Pisciculture

Fish ponds fertilised with community wastewater have been used in India as well as in other countries (2). Extent of fertilisation depends on several factors such as wastewater characteristics, climatic conditions of the region, physical features of the pond and the type of fish reared.

Because of various influencing factors which have not been properly delineated so far the practice of fish culture in sewage fed ponds has been developed by the fish farmers in India, largely on the basis of local experience. Use of raw wastewater for pisciculture is generally associated with a number of management problems and hazards. These can be overcome, to a great extent, if instead of raw waste water treated wastewater is used for pisciculture.

Pond Ecosystem Vis-a-Vis Pisciculture. Organisms present in a pond ecosystem may be classified into three broad groups, namely producers, consumers and decomposers. Producer organisms include phytoplankton, rooted vegetation and large floating plants. Out of the three, phytoplankton are most efficient in synthesing food for the ecosystem because of their large surface to volume ratio. Hence, the productivity of a fish pond is more when management procedures are directed to minimise the other two producer organisms. Biological means of achieving the same is to rear fish which feed on aquatic weeds. Consumer organisms which directly or indirectly consume the producers include zooplankton, insects and fishes. These are further divided into primary, secondary and tertiary consumers depending upon the number of steps through which their food has been obtained from the producers. Bacteria and fungi belong to the decomposer group because they decompose the dead plant and animal cells, as well as the elimination products of living organisms, thereby releasing simple inorganic substances to the environment, for the utilisation by the producers. A small fraction of the solar energy is trapped by the producers and this subsequently flows through the food chain in the ecosystem. A good amount of this trapped energy, however, gets lost at each transfer (3). This ecosystem concept when applied to pisciculture suggests rearing only those species of fish having shorter food chain, ideally the herbivores. This ensures less loss of trapped energy and gives higher fish yield. Since phytoplankton are always associated with zooplankton in a pond ecosystem, fishes which feed both on photoplankton and zooplankton may be raised for better utilisation of the fish food organisms developed in the pond. The stocking density and variety of fish are, however, governed by the quantity and characteristics of plankton and bottom fauna developed. Fishes beyond third tropic level, i.e. secondary consumers are not recommended to run a high yielding pond.

Biotic Community. Phytoplankton production in a sewage-fed pond is generally high because of easy availability of nutrients. Phytoplankton chiefly consists of chlorophyceae, myxophyceae and diatomaceae. The species responsible for algal bloom in a sewage-fed pond are usually *microcystis*, *euglena*, *chlamydomonas*, *chlorella*, *anabaena*, *oscillatoria* and *ankistrodesmus*. Phytoplankton production gives rise to the production of zooplankton which in turn grase over the phytoplankton. This grasing, however, does not cause appreciable reduction in phytoplankton concentration in the system as the number of phytoplankton far exceeds the number of zooplankton. Among zooplankton cladocerans are expected to be abundant and the chief genera usually encountered are *Moina* and *Daphnia*, among copepods, *Cyclops* and *Diaptomus* and among Rotifers *Brachionus*, *Keratella*, *Filinia*, *Asplanchna* are usually present in the wastewater ponds. The bottom of the pond contain bloodworms, sludge-worms, sowbugs, gastropod molluscs and certain fresh water oligochaetes. All these biota are considered to be important fish food organisms (4).

Fish Pond Management. Pisciculture in the tertiary pond of a multiple celled wastewater stabilisation pond is proposed for raising carp from fingerling to table size fish (one kilogram). Hatchling and spawn of carps reared in nursery ponds to fry stage measuring 25-30 mm in two to three weeks time are brought upto fingerling stage in the rearing ponds. These nursery and rearing ponds are not to be fertilised with wastewater stabilisation pond effluent since the physico-chemical and hydro-biological conditions that exist in a treated sewage fed pond may not be favourable for these delicate carp spawn and fries. The management techniques of these ponds are well documented (5),(6). Management details of tertiary wastewater stabilisation pond used as the main fish stocking pond, are discussed under the following heads.

Pond Preparation. The first step towards the preparation of the pond before stocking the desired species of fish is to clear it from weeds manually. This may be done during summer months. The next step is to eradicate all the unwanted fishes particularly the predatory fishes from the pond. The simplest way to do it is to completely drain off the pond and then leave it for natural drying. Alternatively, suitable fish toxin such as endrin at a very low concentration of 0.01 ppm may be applied after partial dewatering.

Species Combination. Composite fish culture may be adopted for obtaining high production of fish. In this practice compatible species of fish having different food habits are stocked together in the same pond. Species combination and stocking ratio in composite fish farming depend on the type and concentration of natural fish food organisms such as bottom biota, zooplankton and phytoplankton present in the pond. Based on the findings of composite culture of carps in different agro-climatic conditions in India carried out by the Central Inland Fisheries Research Institute (CIFRI), Government of India, the following stocking density and species combination have been suggested (7), (8), (9).

L. Rohita (Rohit)	25%
C. Catla (Catla)	10%
C. Mrigala (Mrigal)	15%
H. Molitrix (Silver Carp)	20%
C. Idella (Grass Carp)	15%
C. Carpio (Common Carp)	15%

Figures 2 and 3 depict the above species of carps.

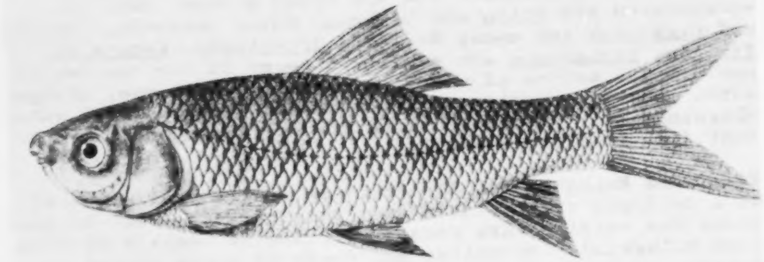


Figure 2(a) L. Rohita

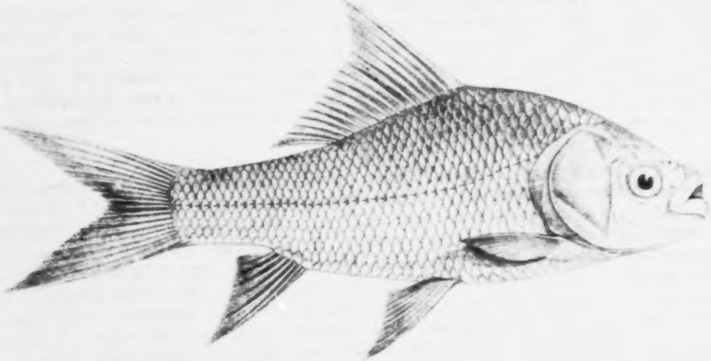


Figure 2(b) C. Catla

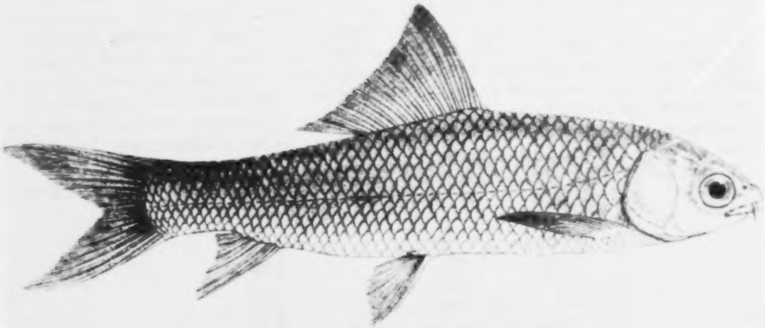


Figure 2(c) C. Mrigala

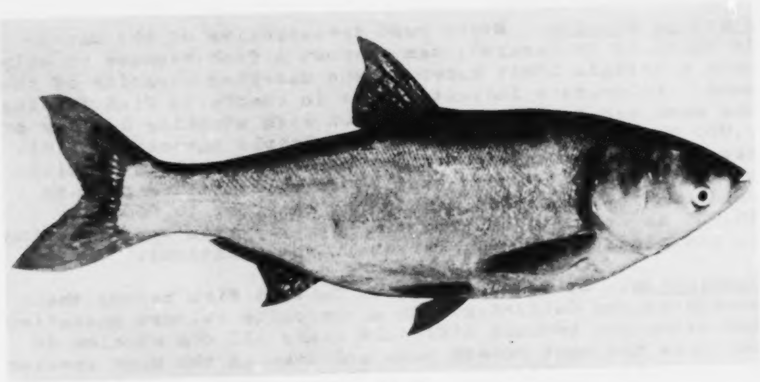


Figure 3(a) H. Molitrix

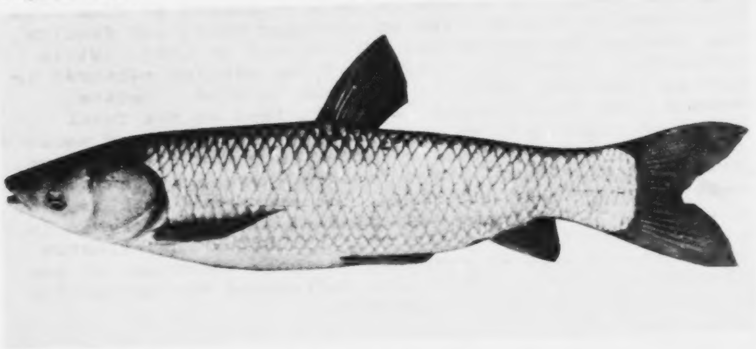


Figure 3(b) C. Idella

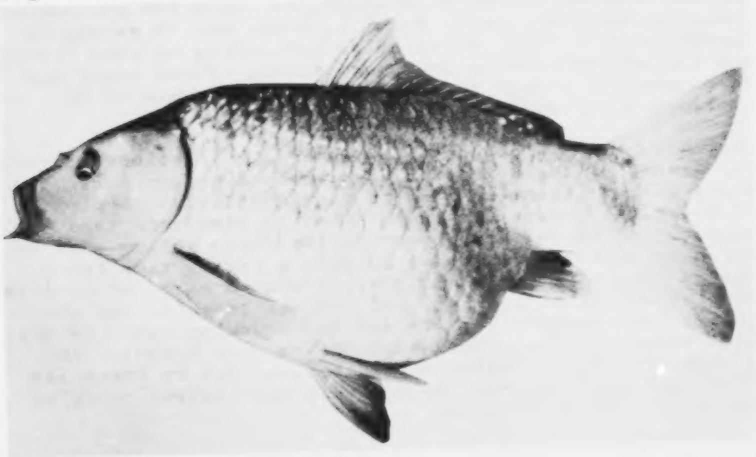


Figure 3(c) C. Carpio

Stocking Density. Every pond irrespective of the manner in which it is managed, can support a fish biomass of only upto a certain limit known as the carrying capacity of the pond. Literature indicates that in composite fish farming, the pond can be suitably operated with stocking density of 4,000 to 7,000 per hectare with multiple harvesting (10). Recent experiments carried out by CIFRI in ponds fertilised with sewage suggest that stocking densities of such ponds may be increased within the range of 10,000 to 15,000 in order to utilise the high plankton concentration in the pond (Ghosh, A., Personal Communication).

Harvesting. It is essential to harvest fish before their growth starts declining. In a composite culture operation, the situation becomes difficult since all the species do not have the same growth rate and even in the same species growth rate is different in different sexes. It is, therefore, suggested that if it is not possible to synchronise the harvesting programme in a community of fish even after careful manipulation of stocking ratio and density, then partial harvesting may be resorted to (10). While harvesting, inter-relationship of the species cultured is also an important point to be borne in mind. Bottom feeders like C.Carpio and C.Mrigala feed on the fecal matter of grass carp which is a voraceous eater of aquatic weeds. Unplanned netting of grass carp would, therefore, affect the growth of bottom feeders whereas if the bottom feeders are totally harvested, the excessive fecal matter of grass carp may cause depletion of dissolved oxygen in the pond water. Sale price of fish, below one kilogram in weight, is less as compared to those which are of one kilogram and above. This also influences the harvesting programme for making more profit.

Management Problems. Several incidental hazards may arise in composite culture operation. These are to be anticipated and corrective measures should be taken for successful management of the pond. Firstly, there may be excessive infestation of weeds which helps harbouring of weed fishes and depletes nutrients from pond water. These weeds may be cleared by either manual or mechanical means or by application of herbicides.

Secondly, algal bloom occuring generally in summer months may cause serious depletion of dissolved oxygen during early morning hours leading to fish mortality. Depletion of dissolved oxygen forces the fish to come upto the surface. When this is observed, immediate action must be taken to save the fish crop by adding oxygen rich fresh water. Thirdly, early maturity of common carp (after four to five months of rearing) and natural breeding may also change the stocking density and the stocking ratio in the pond unless they are harvested before they breed. The number of fish harvested should be replaced by fresh lot of fingerlings in order to maintain the desired stocking ratio of different fishes.

Land Treatment

Renovation of wastewater by land application has been gaining importance in recent years (11), (12), (13). Such practice keeps the waterways clean, gives relief to the demand of fertilisers as well as irrigation water and reduces waste treatment expenditure by creating an avenue for return through the sale of harvested agricultural products. This is an ecologically sound waste management programme since it helps in returning the nutrients present in the wastewater to the place from where they were withdrawn. In view of scanty information regarding land application of treated wastewater in Indian agro-climatic conditions, studies were undertaken with the following objectives -

1. To recommend the wastewater application rate and types of vegetation that can be readily grown with treated wastewater.

2. To examine the quality of leachate joining ground water.

3. To examine the physical and chemical changes occuring in the soil matrix as a result of application of treated wastewater and sludge.

Different types of food or fodder crops may be raised by the application of wastewater alone or in combination with sludge. Following points were considered while selecting the type of crop to be raised -

1. It must suit the agro-climatic conditions of the area where it is proposed to be raised.

2. It should have fairly high yield so that the nutrients can be recycled to the maximum possible extent.

3. It should have high water requirement so that land area to be acquired is less.

4. It should have easy crop management.

Considering the above four factors, two types of crops were initially selected for the study, namely H. Napier grass and Para grass.

Experimental Bed Studies. In the first phase, an experimental bed of brick work was constructed in the back yard of the civil engineering department of Jadavpur University. Soil procured from the University Campus was placed on the bed in such a manner that the collected top soil was retained as the top soil of the bed with successive layers following. The soil was then compacted to the size 2400x1200x1200 mm deep to represent the original soil condition at the site. Suction lysimeters were installed at depths 150, 600 and 1200 mm from the top for the collection of percolate samples. Positioning of the lysimeters at the above depths was done to cover the soil section of three different layers. Soil samples were collected from the three layers and were analysed for different physical and chemical parameters to know the soil characteristics before application of wastewater.

Prior to experimental bed studies pot studies were carried out in 300 mm dia. earthen pots, using the same soil and with two crops, H. Napier grass and Para grass. Objective of the pot study was to observe the crop growth response at different application rates and to select the most suitable rate for a particular soil-crop combination. It was observed that growth response of H. Napier grass was better than that shown by Para grass at all the application rates. Experimental bed studies were subsequently taken up with H. Napier grass. In all eight cuttings were sown at regular spacings. Treated wastewater obtained from Ganguli Bagan biofiltration plant was applied twice a week, the rate being 25 mm per application or 50 mm per week. Crop growth response in the bed was observed and the quality of leachate from different depths was monitored regularly. Leachate samples were collected from 150 mm, 600 mm and 1200 mm soil depths, after each application of treated wastewater on the bed, and were analysed. Average values of the leachate characteristics are presented in Table 3.

TABLE 3
Characteristics of Leachate Obtained From
Different Depths of the Experimental Bed*

Constituents	Soil Depth (mm)		
	150	600	1200
pH	8.23	7.88	7.88
Electrical Conductivity**	2.674	4.137	4.380
Total Dissolved Solids	2565	2605	2627
Ammonia-N	Trace	Trace	Trace
Kjeldahl-N	Trace	Trace	Trace
Nitrite-N	Trace	Trace	Trace
Nitrate-N	3.17	6.13	5.84
Total Phosphorus (P)	0.089	0.243	0.038
Filtrable Phosphorus (P)	0.089	0.243	0.038
Potassium (K)	2.84	3.07	3.01
Chloride	693	1307	1257
Sulphate	79.3	124.7	130.6
Sodium	475	729	645

*Average values reported.
**Expressed as millimhos/cm at 25°C.
Except pH, all values expressed as mg/l.

Irrigation was continued for 93 days and then first cutting of the grass was carried out. Soil samples were collected from the said three layers after the irrigation was terminated and were analysed to find out the changes occured in the soil mass due to application of wastewater. Initial and post irrigation soil analysis results are presented in Table 4.

TABLE 4
Soil Analysis Results

A. Physical

Soil Depth mm	pH 1:2.5, Soil:Water	Electrical Conductivity millimhos/cm		Mechanical Analysis			Textural Class
		1:5 Soil:Water	Saturation Extract	%Sand	%Silt	%Clay	
Initial							
0-150	8.46	0.164	0.556	80.97	3.41	15.62	Sandy loam
150-600	7.60	0.215	1.775	74.28	10.37	15.55	Sandy loam
600-1200	7.74	0.299	1.908	72.12	12.13	15.65	Sandy loam
Post Irrigation							
0-150	9.56	0.265	1.[illegible]5	81.93	3.20	14.85	Sandy loam
150-600	9.40	0.222	1.033	82.39	9.72	10.65	Sandy loam
600-1200	9.92	0.238	0.744	80.32	11.22	8.45	Sandy loam

B. Chemical

Soil Depth mm	Organic C %	Organic Matter %	Nitrogen (N) ppm	Phosphorus (P_2O_5) ppm	Potassium (K_2O) ppm	Calcium (CaO) ppm	Magnesium (MgO) ppm	Cation Exchange Capacity me/100gm
Initial								
0-150	0.228	0.392	336.0	6870.0	5310.0	12300.0	3550.0	5.876
150-600	0.350	0.602	427.0	4580.0	4600.0	16000.0	3990.0	5.936
600-1200	0.222	0.382	310.0	3770.0	4780.0	10500.0	2660.0	5.172
Post Irrigation								
0-150	0.51	0.87	364.0	5150.0	3600.0	7200.0	2200.0	5.77
150-600	0.88	1.51	733.0	3700.0	3990.0	6700.0	2210.0	5.45
600-1200	0.56	0.96	435.0	4710.0	4080.0	7560.0	3080.0	5.12

WASTEWATER RENOVATION SYSTEM PROPOSED AT MALANJKHAND

Activities within the Copper Ore Handling Complex at Malanjkhand produce two major waste streams besides copper tailings. One is the oily wastewater of 100 m^3/day emanating from central repair shop and maintenance garrage and the other is sanitary wastewater of 450 m^3/day emanating from different units including the administrative block. Oily wastewater after treatment in the oil trap-cum-mud settler is mixed with sanitary wastewater and the combined flow after screening and grit removal is pumped to the proposed wastewater reclamation system. Figure 4 shows the system's layout with contours.

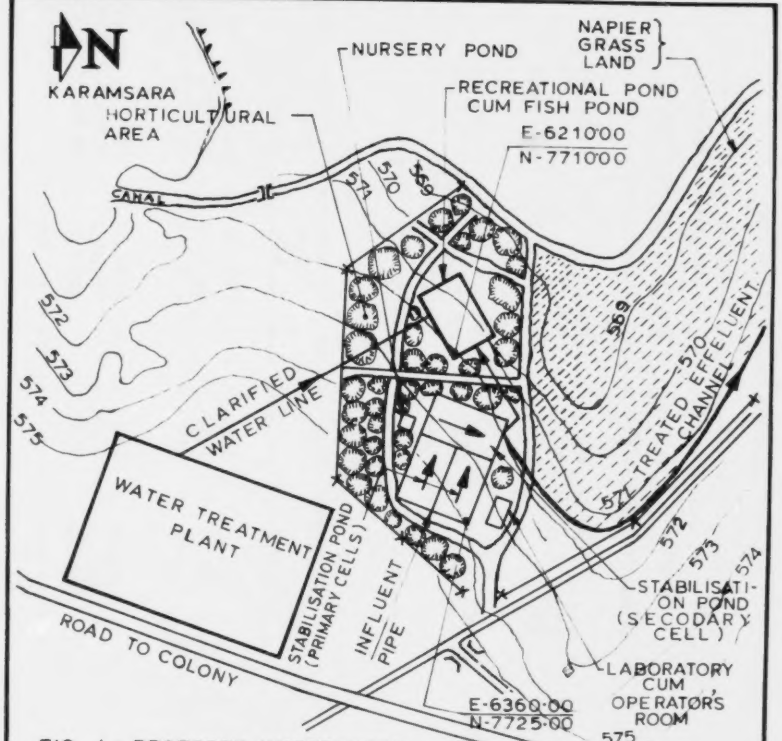


FIG. 4 : PROPOSED WASTEWATER RENOVATION COMPLEX AT MALANJKHAND

Stabilisation pond, the central wastewater treatment unit, comprises two parallel primary cells, each having an area of 60mX30m and 1.5 m deep, followed by a secondary cell of the same size. Wastewater contains good amount of settleable organics which get an opportunity to settle out in the primary ponds and get digested anaerobically at pond bottom. Non-settleable organics on the other hand get stabilised in the upper layer of the primary cells. Stabilisation of dissolved and non-settleable organics is further extended in the secondary cell. The effluent coming out of the secondary cell of the stabilisation pond is divided into two streams, one feeding the recreational-cum-fish pond of area 60mX30m and 1.5 m deep, while the other feeding the effluent irrigation channel. Treated effluent from the irrigation channel will be applied on the land at a rate of 50 mm per week for raising H. Napier grass. Sludge from the Primary Ponds is also to be periodically applied on the grass land along with treated effluent. The nature of top soil of the land to be irrigated is medium stiff yellowish grey silty clay. Mechanical analysis of soil indicates percent gravel, sand, silt and clay and are 2, 12, 36 and 50, respectively. The topography of the area permits the treated effluent to flow by gravity over the grass land before joining the canal originating from the manmade "Karamsara" reservoir. In the reacreational-cum-fish pond six different compatible species of carps will be raised in the manner discussed earlier. Small fries of fish procured from the market are to be brought up to fingerling stage in the nursery pond before releasing the same in the main stocking pond. Recreational activities in the pond will include angling and possibly pedal- boating for children. Water from the fish pond will be used for gardening including raising of flowering trees. A clarified water line from the water treatment plant which is close to the wastewater renovation complex is taken upto the fish pond for supplying dilution water which may be required in case the fishes are dying due to depletion of dissolved oxygen.

SUMMARY AND CONCLUSIONS

Conservation of nutrients present in the community wastewater through aquaculture and agri-horticulture is essential for maintaining the ecological balance. An attempt was, therefore, made to evaluate the agricultural use potential of community wastewater and sludge, elucidate the planning and management details of pisciculture in treated wastewater and to determine the treated wastewater application rate for different soil-vegetation complexes. Studies were carried out with the above objectives using biofiltration plant effluent as irrigant and anaerobic lagoon sludge as soil conditioner. Analysis results of Ganguli Bagan Bio-filtration Plant effluent indicate that total dissolved solids concentration of 3046 mg/l was rather high and this restricts its use to only salt tolerant crops in permeable soils with careful management practice (14). The concentration of trace elements were low which should not pose any problem for its agricultural use. Percent Nitrogen (N), Phosphorus (P_2O_5) and Potassium (K_2O) values of 1.08, 1.41 and 0.94 in the lagoon sludge of the biofiltration plant were rather less. It could, however, be used as a soil conditioner. Observed concentration of trace elements indicate that it should not have any detrimental effect on crop. Regarding pisciculture in treated effluent, composite fish culture technique may be adopted for proper utilisation of fish food organisms developed in a sewage fed pond. The proposed stocking density of fish, species combination and stocking ratio of different species have been discussed. It is rather a short term investigation for prescribing the wastewater application rate for a particular soil-vegetation complex or drawing any conclusion regarding the long term effect of application of treated wastewater on soil as well as the quality of leachate joining ground water table. Studies are being conducted at Jadavpur University, Calcutta (India), using different soil-vegetation complexes with varying application rates of treated wastewater and sludge. However, the author is tempted to comment, on the basis of the results reported here, that there may be an increase in Total Dissolved Solids (TDS) and Nitrate-N (NO_3^--N) of the ground water when such application of wastewater is done without using the sludge. In the present study, about 61% of the applied Nitrogen in the form of NO_3-N and 86% of the applied TDS have been found to leach out of the experimental bed. Removal of other macro and micro-nutrients by soil-vegetation system appears high. It may be pointed out that the concentration of micro-nutrients such as B, Mn, Zn and Fe in the leachate were below measurable values.

The study reveals that an increase in organic matter, pH, total nitrogen content in the soil and a slight decrease in soil cation exchange capacity have occured as a result of the application of wastewater.

Wastewater renovation system proposed for handling the combined sanitary and treated oily effluent of the Copper Ore handling complex at Malanjkhand is a field application of the reported research works. The system results in hardly any discharge of the treated effluent to the surface water body, 'KARAMSARA' Canal, keeping it free from pollution. It provides an opportunity for revenue through the sale of harvested fish and H. Napier grass and is expected to meet its running expenses, if handled properly. The waste renovation complex has been planned in such a way that it serves as a recreational resort to the people in Malanjkhand township besides renovating the wastewater. It also helps in educating the people, particularly the children, visiting the recreational resort about the urban infrastructural facilities like wastewater disposal system, the provision of which are normally taken for granted. Such intangible benefits are absent in conventional wastewater disposal systems, which inspite of their sophisticated machinery, are normally left in oblivion at some remote corner of the township, amid barrenness, receiving visits from none but the plant operators.

ACKNOWLEDGEMENTS

Research works were carried out at Jadavpur University, Calcutta, utilising the Department of Science and Technology, Government of India, Project grant No. DST/BS/70/75. The author wishes to acknowledge the assistance of research team engaged in the project. The author also wishes to acknowledge the opportunity given by the BIL Management in implementing the system at Malanjkhand Copper Project based on the research works.

REFERENCES

1. CHAUDHURI, N., & BASU, A., Ecologically Balanced Community Wastewater Disposal Systems Management for Developing Countries. *Jour. Inst. of Engr(s), (INDIA)*, 56:2:71 (Feb. 1976).

2. JHINGRAN, V.G., *Fish and Fisheries of India*. Hindustan Publishing Corporation (India), Delhi(1975).

3. ODUM, E.P., *Fundamentals of Ecology*. W.B. Saunders Co., Philadelphia (1971).

4. GHOSH, A., RAO, L.H., & BANERJEE, S.C., Studies on the Hydrobiological Conditions of a Sewage Fed Pond with a note on their Role in Fish Culture. *Jour. Inland Fisheries Soc. India*, 6:12:51 (Dec. 1974).

5. LAKSMANAN, M.A.V., Nursery and Rearing Pond Management Techniques and its Economics, Proc. Fish Farmers Development Agency Officials Training in Composite Fish Culture and Fish Seed Production, Central Inland Fisheries Research Institute, Barrackpore, India. (April, 1976).

6. Techniques of Nursery Pond Management.Bulletin, Central Inland Fisheries Research Inst., Barrackpore, India (1974).

7. CHAUDHURI, H., CHAKRABORTY, P.R., RAO, N.G.S., & JENA, S., A New High in Fish Production in India with Record Yields by Composite Fish Culture in Fresh Water Ponds. *Jour. Aquaculture*, 6:4:343 (April, 1975).

8. ALIKUNHI, K.H., SUKUMARAN, K.K., & PARAMESWARAN, S., Studies on Composite Fish Culture Production by Compatible Combination of Indian and Chinese Carps. *Jour. Indian Fisheries Association*, 1:1:26 (1971).

9. SINHA, V.R.P., Third Workshop of All India Coordinated Research Project on Composite Fish Culture and Fish Seed Production, Central Inland Fisheries Research Inst., Barrackpore, India (1975).

10. MURTHY, D.S., Harvest More Fish Through Composite Culture. *Jour. Indian Farming*, 21:4:40 (1971).

11. Wastewater Treatment and Reuse by Land Application, Vol. I and II, U.S. Environmental Protection Agency, Washington D.C., EPA-660/2-73-006 (August, 1973).

12. Proc. Conference on Recycling Treated Municipal Wastewater Through Forest and Crop Land, U.S. Environmental Protection Agency, Washington D.C., EPA-660/2-74-003 (March, 1974).

13. LOEHR, R.C., Land as a Waste Management Alternative, Ann Arbor Science Publishers, Inc. (1977).

14. Water Quality Criteria, Report of the National Technical Advisory Committee, Federal Water Pollution Control Administration, Washington D.C. (1968).

REDUCTION OF POTABLE WATER DEMAND IN THE ARCTIC BY REUSE

D. BROMLEY, M.Eng.
James F. MacLaren Limited
Edmonton, Alberta

A. BENEDEK, Ph.D.
McMaster University
Hamilton, Ontario

INTRODUCTION

As the search for new fossil fuel sources increases so does the need to develop the Arctic. However, major social, environmental and technological constraints have inhibited this development. One problem, in particular, is the limitation in providing adequate servicing for the Arctic inhabitant in an area which is poorly suited to conventional technology.

The major constraints influencing the designing of utility services are:

- severe climatic conditions
- soil and topography limitations
- severe limitations in the assimulative capabilities of receiving streams
- critical shortages of potable water sources

The limitations on water sources and water quality in receiving streams is a great concern when considering water supply and sewage treatment services.

Very little is known about Arctic water quality except that extreme variations in the water characteristics are a common occurrence. Such extremes would not be tolerated by aquatic life found in a temperate climate. The dissolved oxygen (DO) level in rivers and lakes can fall as low as 1 mg/l in the winter. This is far below the accepted limit of 4 mg/l for fish life in temperate climates. The great sensitivity of the Arctic environment lies mainly in this characteristic alone. Although the Arctic's aquatic organisms may be able to survive these low DO levels, no oxygen-dependent organism can survive on zero DO levels. Therefore, any pollutant loading on the water courses would be extremely toxic. In the winter it is not unusual for large rivers to have no flow and, therefore, the rivers have very little dilution capabilities. It is known that during spring flooding, water courses carry an enormous amount of suspended sediment. Thus, aquatic life must not only survive through low DO levels, but also high suspended solids. Finally, the Arctic aquatic organism must be capable of metabolizing at temperatures less than 0°C. Only very hardy species would be capable of withstanding these harsh conditions.

Although the Arctic rivers produce massive runoffs, there is a water supply problem. The coastal plain area is a virtual desert as it receives only 2 inches of precipitation annually. Water does not percolate through the permafrost, hence, groundwater is minimal. Also, there are major variations in both quality and quantity of waters found in this area. It is not abnormal for water conditions to change drastically over one year resulting in both a water supply and a dependency problem.

Most of the coastal plain has a high artificial water table because of the soil's impermeability. This creates abundant swamps with muskeg vegetation. These swamps are a poor water supply source as they have very high levels of organic and iron content. Some communities can obtain their water from ground storage below the permafrost regions. However, there are several problems with this source:

1. there is difficulty in drilling through the permafrost
2. the wells usually freeze
3. the water usually has a high salt concentration (i.e. greater than 2,000 mg/1).

Most lakes and rivers are impossible to use for a potable water source as only large water bodies do not freeze up completely. The best source of water is the scarce alluvial aquifers which are found in alluviums surrounding large water bodies.

Ultimately, in view of the problems noted above, the most pragmatic approach to water supply and disposal may be partial household water recycling. This paper attempts to develop a package sewage treatment system suitable for Arctic application. Indeed, the intent of this paper is to review the development of this system through the design and testing of a pilot plant.

DESIGN OBJECTIVES

The overall design objectives for the sewage treatment system were as follows:

1. The treatment system should incorporate both biological and physical-chemical processes. This combination will ensure adequate and economical treatment at all times. The physical-chemical processes will provide the necessary treatment when the system first starts up. Once the system is in operation and the biological activity has reached an optimum level, the physical-chemical processes can be phased out, resulting in reduced operational costs.

2. The system's concept and operation must be as simple as possible.

3. The system must be housed within a heated containment area. In addition to minimizing the need for costly outdoor piping, the system should be utilized by only a small number of dwellings which are in close proximity to each other.

4. The system should be capable of providing a high quality effluent suitable for reuse as sewage conveyance water within the household. The effluent should have "desired" average COD and SS values less than 20 mg/l and 5 mg/l respectively, and "not to exceed" average values of 50 mg/l and 15 mg/l respectively.

TREATMENT SYSTEM

To satisfy these objectives the chosen treatment system consisted of five components as shown in Figure 1.

(i) an equalization tank

(ii) a combined biological reactor and clarifier (hereinafter called "the reactor-clarifier)

(iii) an adsorption-filtration column

(iv) a disinfection contact chamber

(v) a final storage tank.

In an actual installation the equalization tank would be used to prevent surge flows from entering the treatment system. All household wastewater would be directed to the equalization tank. The contents of the tank would be pumped at a constant rate to the reactor-clarifier.

Within the reactor-clarifier the waste stream would be first exposed to biological treatment in the form of an extended aeration process. If necessary (i.e. at system start-up) coagulant could be added at the upstream end of the reactor. Mixing in the reactor would be provided by coarse air diffusers which would also supply the necessary oxygen for the biological activity. After passing through the reactor the waste stream would enter an integral clarifier which would contain inclined settling plates to improve solids removal. The integral clarifier would be separated from the reactor by an underflow baffle. This arrangement would allow solids to return to the reactor and thereby provide the necessary sludge return. In addition, unlike the conventional separate clarifier unit, the rapid solids return from the integral clarifier should prevent the sludge from going anaerobic.

After the reactor-clarifier the waste stream would enter an adsorption-filtration column. Since the column utilizes downflow hydraulics and activated carbon as its media, it would be capable of providing both filtration and organic adsorption. This obviously improves the water quality and again ensures proper treatment at start-up (i.e. at start-up the dissolved organics would pass through the relatively inactive biological reactor, but would be adsorbed by the activated carbon in the adsorption-filtration column).

Once the waste stream has passed through the adsorption-filtration column, disinfection would be necessary. This would be accomplished by a chlorine contact chamber which essentially is an upflow vertical tube in which chlorine is added at the bottom.

Finally, the treated water would be directed to a final collection tank so that it could be used within the household as sewage conveyance water.

The treatment system would be a package unit located within a heated containment area in or adjacent to one of the dwellings it serves. The minimum operating temperature within this structure would likely be 3-5°C, as this would prevent freezing, while minimizing energy consumption.

Finally, the system would be designed and manufactured in such a way as to facilitate installation. Design features such as common-wall construction, plastic piping, light-weight material, gravity flow, etc. would be used to their maximum benefit.

EXPERIMENTAL

Pilot Plant

During the experimentation the pilot plant consisted of the same five components as identified earlier. The unit was placed in a temperature controlled room for testing. It should be noted that emphasis on design and testing was placed on the two main components of the treatment system; the reactor-clarifier and the adsorption-filtration column.

The pilot plant was designed to treat sanitary waste typical of that produced by one man in an Arctic environment. Using the information obtained from the literature, the following design influent parameters were used:

(i) flow = 45 Igal (200 l)/day
(ii) COD concentration = 700 mg/l
(iii) SS concentration = 500 mg/l.

Experimental Plan

The testing consisted of 8 runs spread over a ten week period. Each run lasted 4 days. The purpose of the testing was to assess the system's capability for treating sewage with:

(i) high - low pollutant concentration
(ii) high - low flow
(iii) coagulant and no coagulant addition
(iv) high - low temperature
(v) effectiveness at start-up after a system shut down.

Operating conditions for each run were as follows:

run 1 - COD - 1200 mg/l
Flow - 73 Igal (330 l)/day
Temperature - 4°C

run 2 - COD - 400-500 mg/l
Flow - 45 Igal (200 l)/day
Temperature - 4°C

run 3 - COD - 1200 mg/l
Flow - 45 Igal (200 l)/day
Temperature - 4°C

run 4 - COD - 400-500 mg/l
Flow - 73 Igal (330 l)/day
Temperature - 4°C

run 5 - COD - 400-500 mg/l
Flow - 45 Igal (200 l)/day
Temperature - 4°C
* no coagulant added

run 6 - COD - 400-500 mg/l
Flow - 45 Igal (200 l)/day
Temperature - 15°C

7 DAY SYSTEM SHUTDOWN

run 8 - COD - 400-500 mg/l
Flow - 45 Igal (200 l)/day
Temperature - 4°C

Daily pilot plant performance was measured by three parameters:

(i) chemical oxygen demand (COD)

(ii) dissolved chemical oxygen demand (dissolved COD)

(iii) suspended solids.

RESULTS AND DISCUSSION

Treatment Capabilities

The overall system's performance (i.e. the system's influent compared with the final effluent) is displayed in Figures 2 through 5.

The results indicated that:

(i) generally, although there were different influent COD and SS concentrations and flow rates, there were no statistical differences in the final effluent with regard to the parameters monitored.

(ii) the average COD removal for the overall treatment system was 94.3 percent, with an average COD concentration in the final effluent of 37.2 mg/l.

(iii) the average SS removal for the overall system was 96.4 percent, with an average SS concentration in the final effluent of 13.8 mg/l.

(iv) 81 percent of all COD concentrations in the final effluent were below the "not to exceed" objective of 50 mg/l and 41 percent of the COD concentrations were below the "desired" objective of 20 mg/l.

(v) 78 percent of all SS concentrations in the final effluent were below the "not to exceed" objective of 15 mg/l and 34 percent of all final effluent SS concentrations were below the "desired" objective of 5 mg/l.

(vi) the average COD and SS removal in the reactor-clarifier was 78 percent and 83.4 percent respectively resulting in an average COD and SS effluent concentration from the reactor-clarifier of 143 mg/l and 68 mg/l respectively.

(vii) for the adsorption-filtration column the results indicated that the average removal of the column's influent COD and SS removals were 75 percent and 80 percent respectively from the columns influent concentrations.

The average dissolved COD removal in the adsorption-filtration column was 76.3 percent which resulted in a final effluent average concentration of 7.6 mg/l. In addition through the analysis of isotherms produced on the used and clean activated carbon, it was estimated that approximately 57 percent of the removed dissolved COD was through biological activity on the surface of the activated carbon and 43 percent was through adsorption.

It was noted that for future designs of a similar nature 11.5 pounds of removed COD/100 pounds of activated carbon should be used as a design parameter.

Feasibility of Recycle

Calculations were made on the resulting concentration increases of COD and SS in the final effluent if both backwash water from the adsorption-filtration column and final effluent are recycled back into the system. It was assumed that final effluent from the system would be reused as sewage conveyance water within the household (i.e. toilet water).

On the basis of these calculations it was evident that if treatment predictions were based on the average values and the average values (i.e. removal percentages) do not change when backwash water and final effluent are recycled to the head of the system, then the SS concentration in the effluent should become constant after day 4 at a level of 12 mg/l.

Similar results were found with COD concentrations. Here, however, COD values did not level out until day 6, when the concentration reached 39 mg/l. These levels would satisfy our design objectives. One should not expect in an actual system such a rapid approach to equilibrium as indicated, but eventual equilibrium is probable. Nonetheless, if this level is very high for either SS or COD, then recycle of backwash and/or final effluent would be unacceptable for aesthetic and/or hygienic reasons. From the results of these calculations it would seem that recycle is feasible.

SUMMARY

A sewage treatment system was developed for Arctic application. The pilot plant of this system was constructed and tested. One of the main objectives of the package system was to produce a high quality effluent which could be recycled back to the household as sewage conveyance water. This objective was important due to the severe shortage of potable water sources and the sensitivity of receiving streams in the Arctic.

From the testing of the pilot plant it was found that:

1. The system operated well as a treatment process. Generally, the system provided consistent final effluents and was capable of treating wastes with high organic and solid concentrations such as those concentrations employed during the testing procedure.

2. The system operated well at start-up. The use of coagulant ensured good treatment in the reactor at the beginning of the testing period as did the activated carbon in the adsorption-filtration column. After one month, when the biological activity was at an optimum

level, the overall system did not require coagulant addition, Also, it was shown that the biological activity recovered rapidly after the system was shut down for 7 days. The physical-chemical and biological aspects of the system worked well together to ensure consistent and efficient treatment.

3. Based on predictive calculations, recycle of final effluent as sewage conveyance water and backwash water for the adsorption-filtration column seemed feasible.

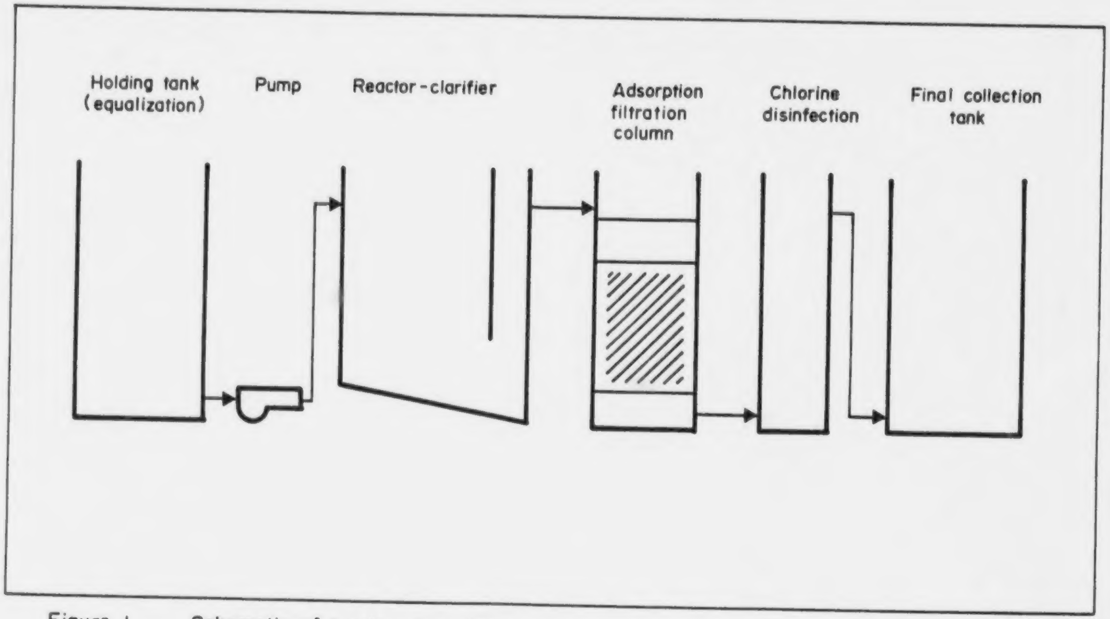


Figure I Schematic of treatment system.

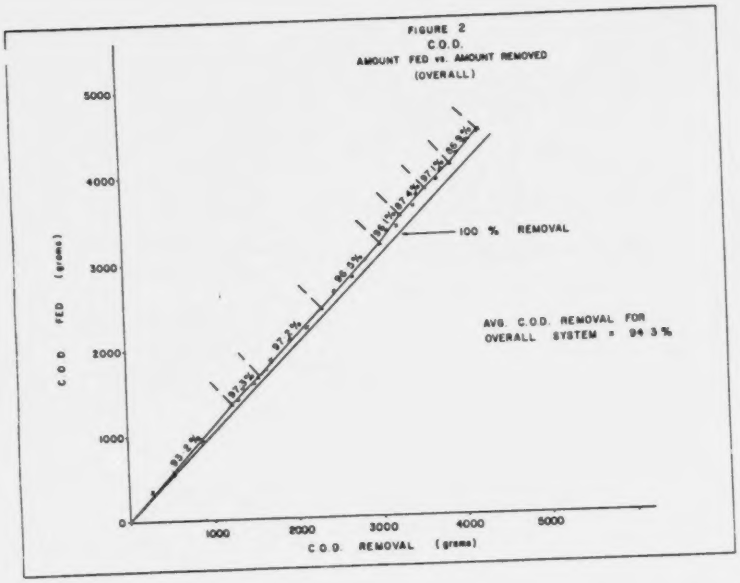


FIGURE 2
C.O.D.
AMOUNT FED vs. AMOUNT REMOVED
(OVERALL)

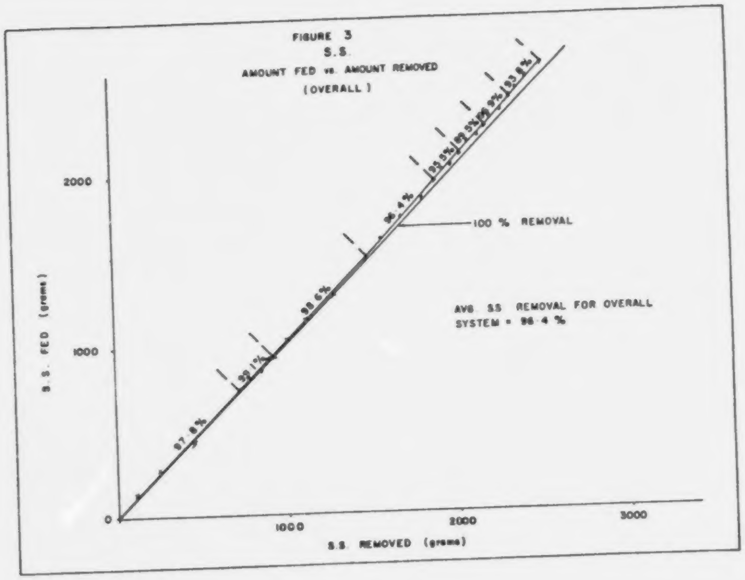


FIGURE 3
S.S.
AMOUNT FED vs. AMOUNT REMOVED
(OVERALL)

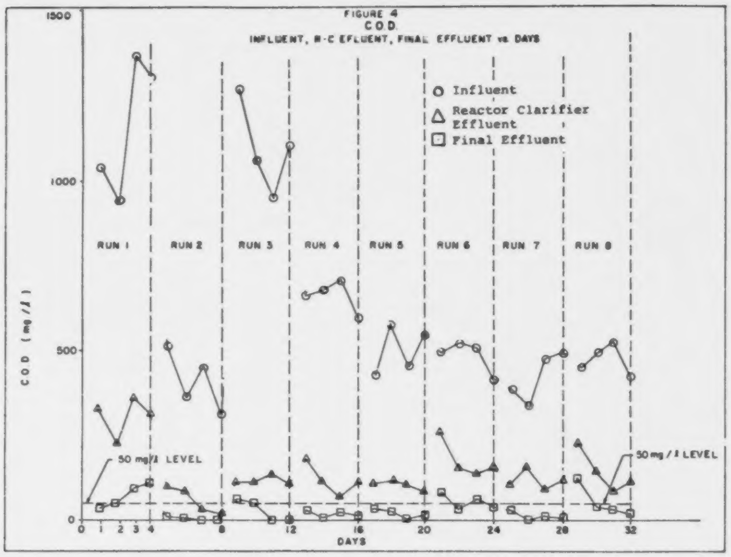


FIGURE 4
C.O.D.
INFLUENT, R-C EFFLUENT, FINAL EFFLUENT vs DAYS

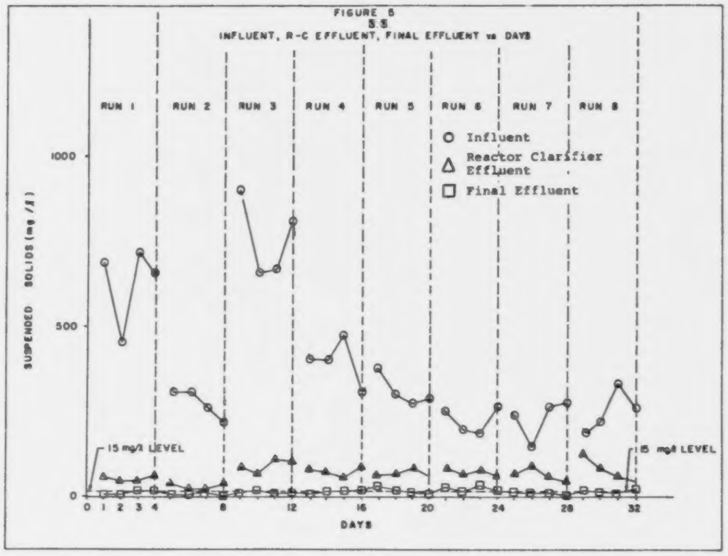
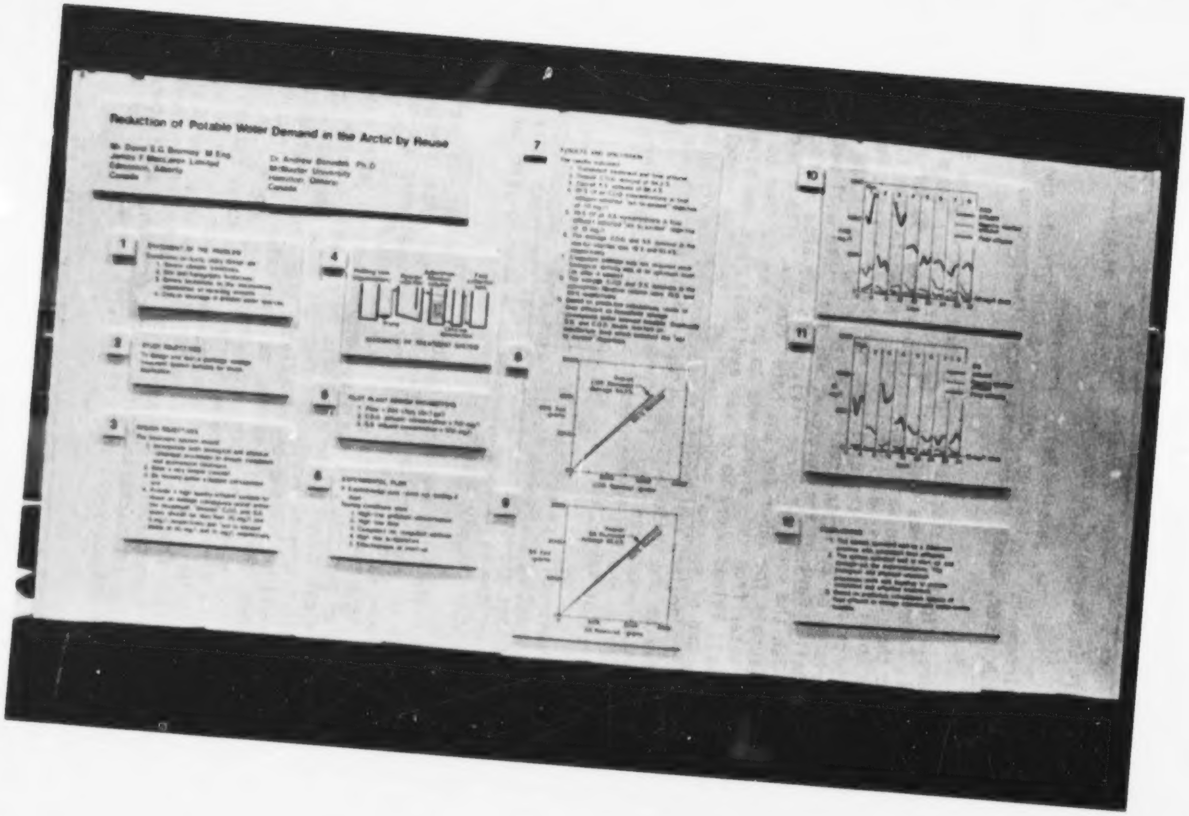


FIGURE 5
S.S.
INFLUENT, R-C EFFLUENT, FINAL EFFLUENT vs DAYS



Reduction of Potable Water Demand in the Arctic by Reuse

Mr. David E.G. [illegible] M.Eng
James F. MacLaren Limited
Edmonton, Alberta
Canada

Dr. Andrew Benedek Ph.D
McMaster University
Hamilton, Ontario
Canada

WATER RECYCLING - NECESSITY AND A PROFIT

Dr. John R. Snell, President
Snell Environmental Group, Inc., Lansing, Michigan

Dr. Sachiho Naito, President
Environmental Sanitation Engineering Consultants and
Overseas Advisor, Japan Water Works Association, Tokyo, Japan

In 1973, hydrological studies for a 6,400 room hotel complex on the Isle of Bali, Indonesia, was completed by international hydrologists. These showed that adequate water was available from a 90 km^2 peninsula on which the hotel complex was to be built. The World Bank financed plan was to provide all of the utilities and the facilities for twelve private hotels to be built in specific areas along one of the finest beaches in Bali. Part of the agreement was that this well water was also to furnish the needs of three villages in the area which, in turn, would supply the labor for the hotel complex. The alternative to securing water from this source was to go 30 km further to the main portion of Bali Island where water was available, but was already in short supply and of somewhat poorer quality. The prime consulting engineer on the project was Pacific Consultants of Tokyo; but the water, wastewater and solid waste portion of this project was handled by a joint venture between Snell Environmental Group, Inc. (SEG) of the United States and the Environmental Sanitation Engineering Consultants (ESEC) of Tokyo, Japan.

Ground water hydraulic calculations and recharge calculations made by SEG and confirmed by ESEC, and finally by a third independent hydrologist, showed that approximately 25 per cent of the fresh water needs of the complex and the villages would exist on a year-to-year basis from the proposed water supply source.

After considerable study and consultation with our clients, it was determined that the same source would be useful, but that water needs would be reduced by building only half of the hotel complex--namely, 3,200 rooms, thus reducing the water requirements from 15,983 m^3 per day to 4,090 m^3 per day. It was also determined that water needs beyond what was determined to be available in the well complex would be met by recycling treated wastewater from the whole complex. The recycled water would be limited to non-potable uses including: walk and street washdown, grounds irrigation, make up water for air-conditioning cooling, make up water for refrigeration cooling, and for the central laundry. These five areas were estimated to use about 55% of the total water needs of the complex. It was determined not to use recycled water for toilet flushing within the building so as to avoid possible future cross connections. The quality of the water after treatment, both chemically and bacteriologically, met drinking water standards; however, for the above-mentioned reasons, it was not used for potable purposes.

The raw water contained some iron and 370 mg/L of total hardness. A lime soda softening plant is used for the overall supply. One of the big economic advantages of recycling was that the recycled water was already softened and did not need to be re-softened. The cost of softening was more than the cost of wastewater treatment and sterilization. This system did require a two-pipe, outside system--one for recycled water and one for the original softened water. There was also a local well which produced lower quality water (that is, high in iron and hardness) which will be used as a standby in case of emergency for either the potable or the recycled water system. The primary water source was in the center of the peninsula in porous limestone wherein 11 wells were drilled, penetrating the freshwater aquifer approximately 76 m.

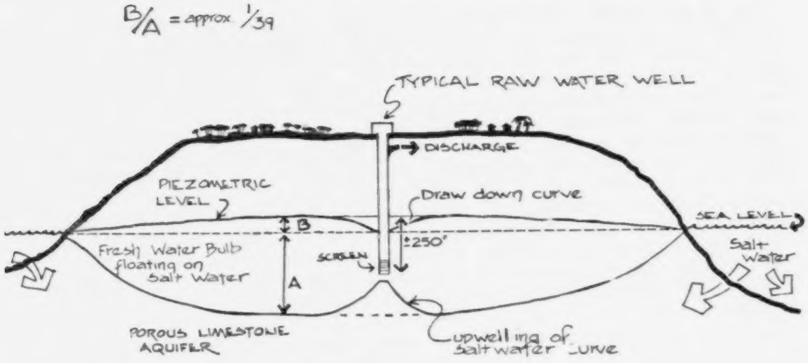
Figure 1 illustrates a cross section of the piezometric level of the lense of fresh water floating over the salt, and also illustrates the effect of a typical well pumping in the aquifer. This illustrates both a drawdown curve and an upwelling curve of salt water on the bottom. The rainy season, which lasts less than four months, brings about the recharge of the freshwater aquifer. Hence, the freshwater lense must last seasonally as the only source of water supply. Initial hydrologic calculations did not fully consider movement of the salt water. Recharge limitations from rainfall limited the water available.

A large marl tidal flat, located northeast of the property, was acquired for the design and construction of a four-cell sewage purification lagoon. The raw wastewater was pumped to the lagoon at a fixed point, and flowed through the four cells prior to being either recycled or periodically discharged into deep water estuary. Within the area of the lagoon, there was considerable shallow ground water which normally seeped into the estuary and mixed with the salt water. In order that the fresh water may be captured, a special ditch was excavated between the lagoons. This served two purposes. First, to keep the lagoon water from contaminating the ground water, because the lagoon surfaces would be higher than the normal ground water. The second purpose was to capture the surplus ground water itself before it escaped into the bay. This shallow ground water would thus constitute the first priority use of recycled water before further treatment by coagulation, filtration and chlorination. The second priority use of water before further treatment would be that seeping from the No. 4 lagoon through a porous dike. The third and last priority wastewater would be water from the No. 4 lagoon itself.

The lagoons were unique in design in order to greatly reduce the leakage. The three feet of marl in the lagoon area over limestone was first compacted in the dry to prevent leakage. Another unique feature was that carp are to be grown in the first two lagoons and non-bottom feeding fish in the last two. All of these fish will be commercially grown and harvested.

The mixture of shallow ground water and wastewater will be treated with alum, mixed, coagulated, settled, filtered and chlorinated. Settling will take place in open basins which will be drained periodically to allow the sludge to dry for easy removal. Filtration will be provided by other multi-media gravity filtration units. The chlorinated effluent will be piped to a central reservoir and distributed separately from the primary potable water supply.

Construction of this wastewater reclamation and reuse system is now underway and almost completed, and arrangements are being made for construction of several hotels. It is fortunate that SEG engineers sensed an inadequacy in the initial hydrology study, and went beyond the scope of their work to check the results. Otherwise, a series of grave consequences could have taken place, including the possible construction of $30 to $50 million of hotel rooms which could not be furnished water from the available well field. As it turned out, no harm was done and with the developing fuel crisis, fewer hotels will now be needed. By using recycling to make up the remaining difference, the cost of recycling was less than the initial cost of softening; hence, the recycling system turned out to be not only a necessity but profitable as well. The fish which will be harvested may be looked upon as a further fringe benefit in the profit margin.



TYPICAL CROSS-SECTION OF ISLAND

A WATER REUSE PROGRAM ON ST. CROIX, VIRGIN ISLANDS

O. K. Buros, Ph.D.
Water Resources Engineer
CH2M HILL, Inc.
P.O. Box 1647
Gainesville, FL 32602

INTRODUCTION

The U.S. Virgin Islands is an unincorporated territory of the United States of America. It is located about 2,000 km (1,200 miles) southeast of Miami, Florida. The Territory is made up of three major islands, St. Croix, St. John, and St. Thomas, plus about 50 smaller islands and cays. The total land area is about 35,000 ha (135 square miles). St. Croix is the largest island, with an area of 21,800 ha (84 square miles) and a current population of approximately 48,000.

Water Supply--1960's

In the mid-1960's, less than half the homes on the island were connected to a potable water distribution system. The homes not on the system, as well as many on it, used rainwater collected on their roofs and stored in individual cisterns which were built into the homes. In 1968, the distribution system was improved to connect the two major towns of Christiansted and Frederiksted to a unified distribution system, which was supplied by several well fields. In addition, the two towns had a dual distribution system, which supplied sea water for fire fighting and sanitary flushing purposes.

Although St. Croix receives an average of 1,000 mm (40 inches) of rainfall annually, the soils, geology, and climate seriously restrict the amount available as ground water. The evapotranspiration rate is extremely high and the aquifer storage capacity is relatively low. Droughts which occur every few years periodically reduce the yield of the wells on the island, thus restricting the dependable quantity of water available for the distribution system.

Development--1960's

During the 1960's, the islands experienced unprecedented growth. The economy diversified from predominantly agriculture to one which emphasized tourism and industry. Between 1960 and 1970, the population doubled, putting a tremendous strain on the existing water system and available ground-water supply.

In the late 1960's, the Virgin Islands Government moved in two significant but different directions toward solving their long-term water resource problems. These solutions were to increase the available supply by using sea water desalination and wastewater reclamation. In retrospect, these decisions seem to have been sound, but both paths, although full of potential, have had their share of problems and frustrations.

Desalination

In 1968, the first desalination plant went into production, a second began producing in 1975, and the Virgin Islands Water and Power Authority (VIWAPA) is currently preparing to purchase a third. These are dual purpose plants which use the distillation process to produce freshwater. The VIWAPA was among the early pioneers in the large scale use of land-based sea water desalination plants, and their plants have had their share of problems. Corrosion, scaling, and mechanical breakdowns have caused continual serious problems with the operation, and the average production is about 60% of nameplate capacity. Early water production cost predictions for desalination of \$0.25 to \$0.45/m^3 (\$1.00 to \$1.75/1,000 gallons) did not materialize and current costs for desalinized water from various plants on the island range from \$1.30/m^3 to \$5.00/m^3 (\$5.00 to \$20.00/1,000 gallons).[1]

WATER REUSE PROGRAM

The idea of water reuse was prominantly mentioned in a report published in 1968.[1] Various methods were discussed, including ground-water recharge. In 1970, the concept of reuse finally took shape with the encouragement of two federal agencies, the U.S. Geological Survey and the Federal Water Pollution Control Administration (later reorganized as the U.S. Environmental Protection Agency). The USGS suggested using treated wastewater for ground-water recharge and the FWPCA supplied funds for a project through partial funding of research and construction costs.[2,3]

Wastewater Reclamation Project

The concept was supported by the Virgin Islands Government, which allocated matching funds and administrative supervision to the original project, marking the beginning of the reuse program which is still in operation. The original project, funded by FWPCA, included a study and the construction of a facility to be used to feasibility of increasing the freshwater reserves on St. Croix by artificially recharging the ground water with tertiary treated wastewater.

The project was placed under the direction of a consultant, Black, Crow and Eidsness, Inc. (now CH2M HILL, Inc.), who implemented the study, design, and operation of the reclamation facility until mid-1975.[3]

During this period, the following tasks were accomplished.

1. Design and construction of a 1,890-m^3/day (0.5-mgd) reclamation plant.
2. Selection of a ground-water recharge site and construction of recharge facilities (spreading basins).

3. Training and supervision of local personnel to operate the plant and recharge facilities.

4. Operation of the plant and recharge facilities for about 2 years.

5. Monitoring ground and surface water in the area.

Reclamation Plant. The reclamation plant consisted of an activated sludge process followed by chemical coagulation, filtration, and chlorine disinfection. It treated primary effluent from a newly constructed primary plant located at the same site on the island's south shore. See Figure 1. The primary effluent not used was discharged via an outfall into the coastal waters.

Recharge Facility. The effluent from the reclamation plant is pumped about 2,400 m (8,000 ft) to the recharge area. Recharge ponds (spreading basins) are used to introduce the treated effluent into the soil system for additional treatment and eventual augmentation of the ground water. The recharge ponds are designed to operate on an alternating wet and dry cycle to promote efficient treatment and infiltration. The layout of the recharge area is shown on Figure 2.

Operation. In mid-1975, after directing the project for 4-1/2 years, the consultant transferred the responsibility to the Government of the Virgin Islands, which has operated it ever since through its Division of Natural Resources Management.

A graphical representation of the quantity of effluent recharged during the program to date is shown on Figure 3.

Operational Problems. Although there were miscellaneous mechanical difficulties in operating the plant and recharge facilities, the three major problems have been due to outside influences. These problems have been:

1. Insufficient wastewater.
2. Storms.
3. Sea water fire and flushing systems.

Concommitant with the construction of the reclamation facilities, the Government was completing the primary plant and outfall at the same site and beginning construction of interceptor lines. These interceptors were to transport the wastewater from the three service areas, shown on Figure 1, into the primary plant. The mid-island area was connected first, collecting wastewater with a moderate chloride content (400-500 mg/l). The salt content of the wastewater was marginally satisfactory for recharging the aquifers whose background chlorides were already about 300 mg/l, but the quantity was originally insufficient for optimum operation of the reclamation plant. However, the flow continuously increased as more and more connections were made to the system.

In October and November 1974, the interceptor system was seriously damaged by a high amount of rainfall and subsequent flooding. This disruption of the wastewater supply to the primary plant caused a shutdown in operation of the reclamation plant.

Immediately following this, the interceptors were connected to the Frederiksted collection system, which contained a high percentage of salts due to the use of a separate sea water sanitary flushing system in town. This caused a further delay in operation until the Frederiksted sea water system was eliminated in February 1976.

Operation resumed for 1-1/2 years until the last major interceptor was completed to Christiansted, at which time a repeat of the Frederiksted problem occurred. Thus, in July 1977, the recharging of ground water was halted due to the high TDS of the effluent.

Current Status. The amount of sea water used in Christiansted is about four to five times that previously used in Frederiksted and thus, the cost to provide an alternative low TDS source is considerably higher. To date, March 1979, the sea water system is still being used. Hence, the reclamation project has been relatively unproductive, except for plant maintenance and monitoring for the past 21 months.

To permit resumption of reclamation and recharge, the intake structure of the primary plant is being altered to allow a portion of the low TDS wastewater from the mid-island and Frederiksted service areas to be bypassed around the primary plant to the reclamation plant. The remaining wastewater, including that containing the high percentage of sea water, will continue to be treated in the primary plant and disposed of by an outfall on the south shore.

The bypass should be completed in April 1979 and the reclamation and recharge facilities should be operational during the summer.

Other Reuse Activities

Several other reuse activities have taken place during the years since the reclamation plant began operation in 1974. These include pisciculture (with *Talapia*), clam culture, pasture irrigation, and small-scale gardening and nursery irrigation. Although some of these activities showed promise, they have become inactive due to the uncertainty of the supply of reclaimed water.

It can be expected that interest in some of these activities will be resumed once it is demonstrated that there will be a dependable supply of reclaimed water available.

DESALINATION AND WATER REUSE

Although the Government appeared to set off in the two different directions of sea water desalination and water reuse in the late 1960's, this combination is actually very complementary. When sea water is desalinated by distillation, it can easily produce a potable water with a TDS below 10 mg/l. This low TDS water can be effectively blended with higher TDS ground water to produce a very potable product. When a low TDS water is utilized in the distribution system, it in turn produces a low (or moderate) TDS wastewater which is essential for long-term recharging of aquifers if salt contamination is to be avoided.

By manipulating the distribution and mixing of desalinated water and ground water within a system such as exists on St. Croix, it is possible to regulate the TDS of the wastewater in the appropriate service areas to minimize the salt content of the reclaimed water and yet provide an acceptable water to all of those on the distribution system. A plan to permit this has been formulated, but it will require a coordinated capital expenditure program.[5]

If the newer sea water reverse-osmosis process was used to produce potable water, it would not give the same advantage. The water from the reverse-osmosis process will generally have a salt content of about 450 to 500 mg/l rather than the under-10 mg/l experienced with distillation plants.

THE FUTURE OF THE REUSE PROGRAM

As with sea water desalination, the reuse program on St. Croix has encountered numerous frustrations and problems. Although many (inadequate flow and sea water interference) were anticipated, it has taken time and patience to achieve solutions within the limitations inherent in the governmental structure and budget.

There are several good reasons why the reuse program can be expected to continue and expand on St. Croix. These include public acceptance, cost effectiveness, stability, and commitment.

Public Acceptance

There has been no real problem in the Virgin Islands as far as public acceptance of water reuse is concerned. The endemically short supply of water has made the public conscious of the necessity for reuse. Wastewater has been reused for various purposes in many homes, hotels, and condominium complexes in the Territory. If reclaimed wastewater was available for use in agricultural applications, it would be readily used by the public.

Cost Effectiveness

With the available potable water costs aside from the limited ground water ranging upward from about $1.30/m^3 ($5.00/1,000 gallons), it becomes very cost effective to treat effluent for reuse for certain applications. Augmenting the ground water and protecting it from saltwater intrusion by ground-water recharge would probably be the most cost-effective application. In 1976, the cost of reclamation, recharge, and recovery was estimated at about 0.57/m^3 ($2.15/1,000 gallons) using the existing facilities.[3] Even with inflation, this cost compares very favorably with alternatives to ground water and can be substantially reduced by improvements to the reclamation.

However, the amount of additional ground water which can be produced by recharge and utilized within the potable distribution system is limited by the transmissibility of the aquifers, the level of salts desired in the distributed water, and the ability of the existing system to blend waters of varying qualities.

There are other uses for reclaimed water, such as irrigation, sanitary flushing, etc., but these have a lower benefit-cost ratio than does ground-water augmentation. Overall, there is a limit to the total amount of reclaimed water that can be economically utilized. This capacity needs to be determined to properly plan future expansion.

Stability

With some alterations to the bypass now being installed, the reclamation plant should be assured of a suitable and stable wastewater supply for future operations. This will produce the overall flow pattern shown on Figure 4. This pattern permits the reuse of water for ground-water recharge and augmentation, but it reduces the chance for the buildup of salts, organics, etc., in the distribution system by allowing a portion of the system's water to be lost after usage by discharge to sea via the primary plant and outfall.

By retaining the dual sea water system in Christiansted, the Government is able to continue to reduce their overall consumption of potable water by about 15%. This is important at this time as the potable water system is restricted in its ability to supply water.

Other suggestions have been made to construct a larger reclamation plant and utilize the effluent for sanitary flushing and extensive irrigation as well as for ground-water recharge.[6]

Commitment

The Government has shown its commitment to water reuse by its continued support of the program through the rather trying and lengthy shutdown period which is now coming to an end. Aside from resuming operation, efforts will be needed to determine what, if any, additional treatment is needed for the recovered ground water and to integrate the reuse system into the island's water resource program. One aspect of this integration will be continued emphasis on the economic evaluation and management of the total water system on St. Croix.

AUTHOR

Dr. Buros is a specialist in the development of water resources in water-short areas. He lived in the Virgin Islands for 5 years while he directed a variety of water- and wastewater-related studies, including the St. Croix wastewater reclamation project.

REFERENCES

1. ENGINEERING SCIENCE, INC. Water Reclamation for the U.S. Virgin Islands. Arcadia, California. (1968).

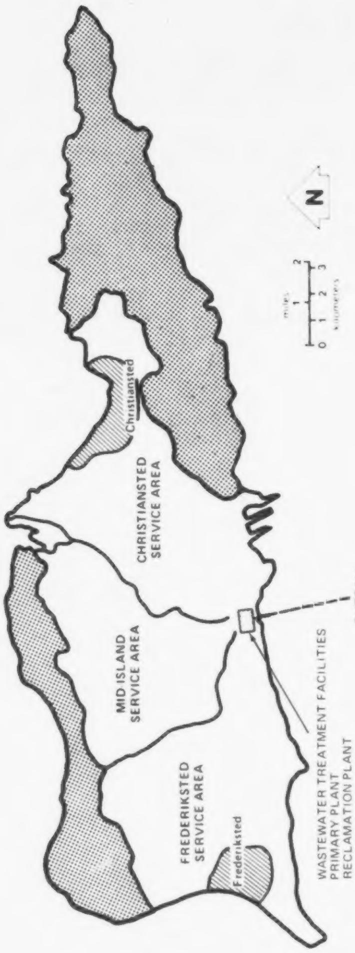
2. ROBINSON, T. M. Ground Water in Central St. Croix, U.S. Virgin Islands. USGS Caribbean District Open-File Report. (1972).

3. BUROS, O. K. Wastewater Reclamation Project, St. Croix, Virgin Islands. EPA-600/2-76-134, U.S. EPA. Cincinnati Ohio. (1976).

4. BUROS, O. K. Water Reuse and Conservation in the Virgin Islands. Desalination, 23. (1977). pp. 485-494.

5. BLACK, CROW AND EIDSNESS, INC. A Water Management Plan for St. Croix, U.S. Virgin Islands. Black, Crow, and Eidsness, Inc. Gainesville, Florida. (1976).

6. CAMP, DRESSER & MCKEE, INC. Facilities Planning Report on Wastewater Reclamation and Reuse for St. Croix. CDM. Boston, Massachusetts. (1976).


CHRISTIANSTED SERVICE AREA
Christiansted
MID ISLAND SERVICE AREA
FREDERIKSTED SERVICE AREA
Frederiksted
WASTEWATER TREATMENT FACILITIES
PRIMARY PLANT
RECLAMATION PLANT
OUTFALL
N
miles
0 1 2
kilometers
0 1 2 3

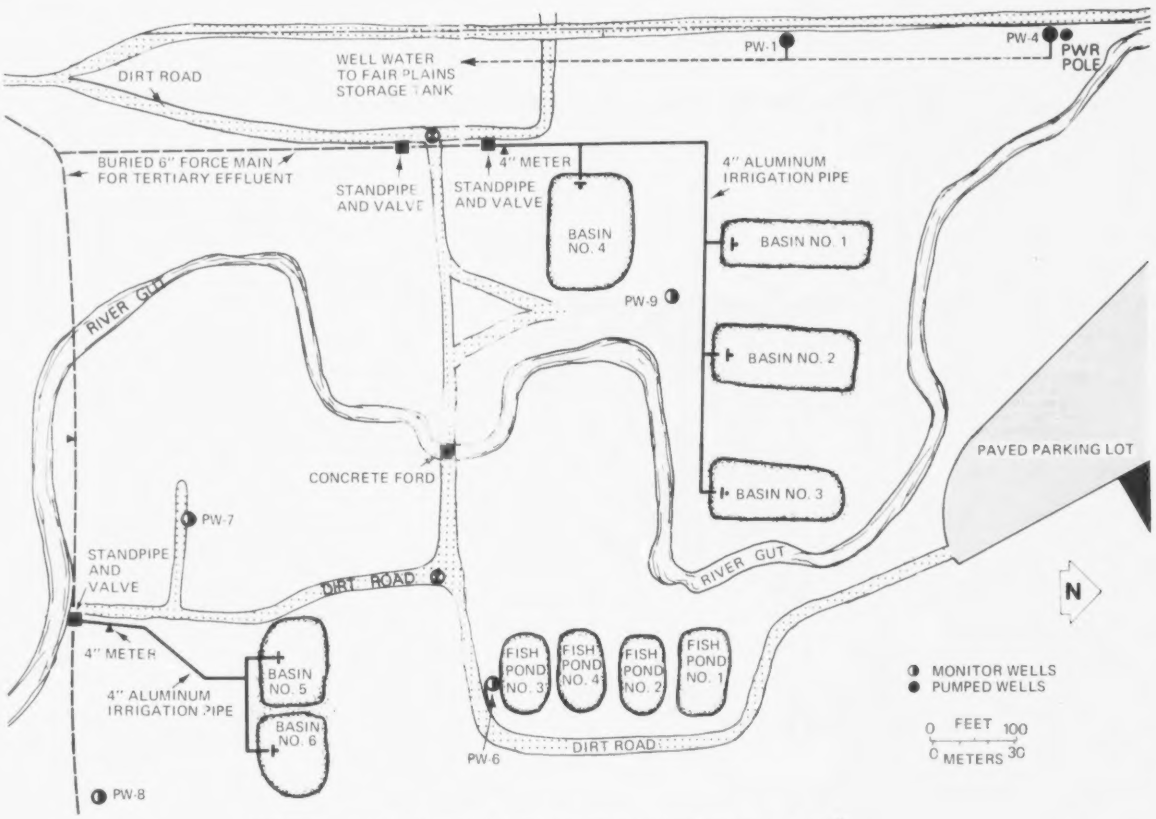


FIGURE 2. The ground-water recharge facilities at Golden Grove on St. Croix.[3]

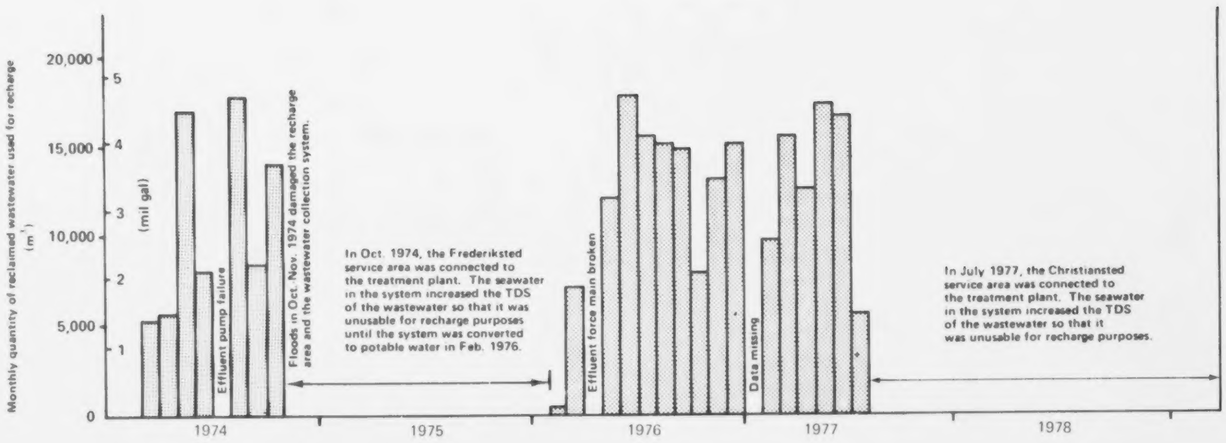


FIGURE 3. Quantity of reclaimed wastewater used for ground-water recharge in the St. Croix project, 1974–1979.[4]

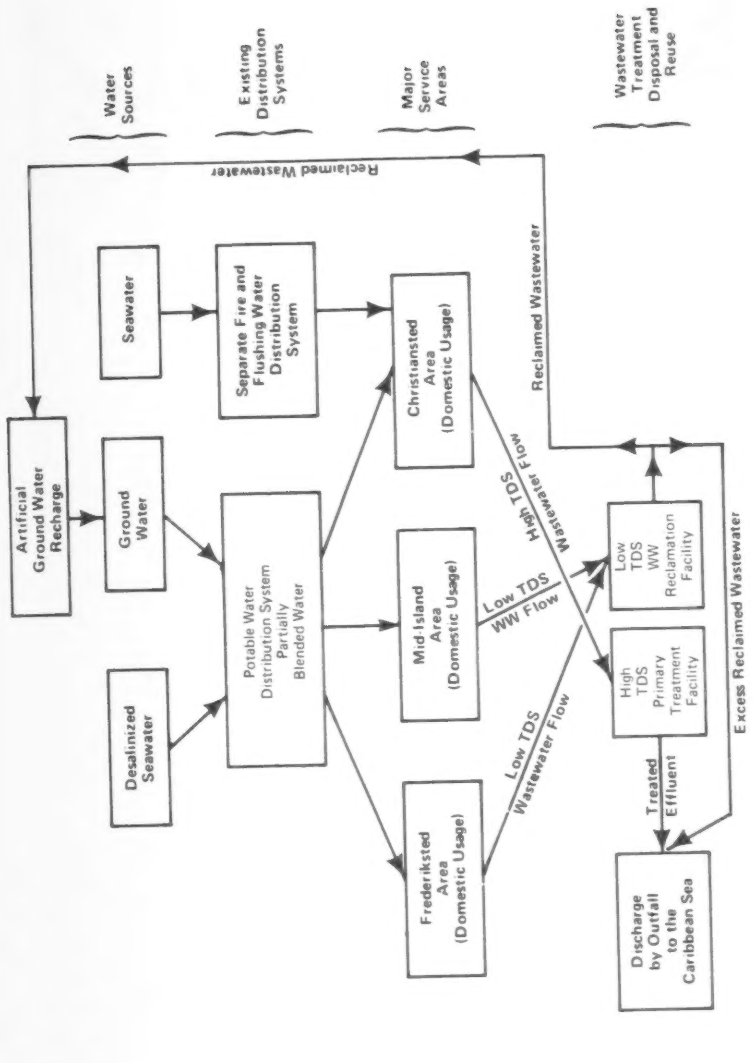
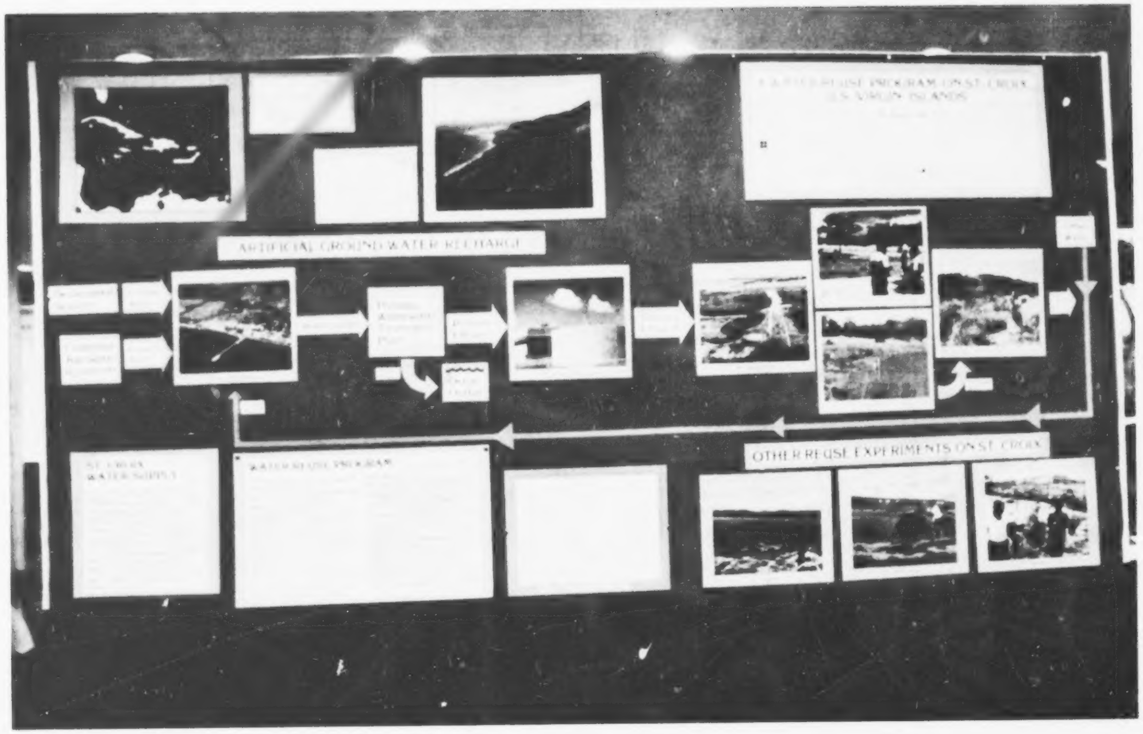

Water Sources
Existing Distribution Systems
Major Service Areas
Wastewater Treatment Disposal and Reuse
Artificial Ground Water Recharge
Desalinized Seawater
Ground Water
Seawater
Potable Water Distribution System Partially Blended Water
Separate Fire and Flushing Water Distribution System
Frederiksted Area (Domestic Usage)
Mid-Island Area (Domestic Usage)
Christiansted Area (Domestic Usage)
Low TDS Wastewater Flow
Low TDS WW Flow
High TDS Wastewater Flow
Reclaimed Wastewater
Reclaimed Wastewater
High TDS Primary Treatment Facility
Low TDS WW Reclamation Facility
Treated Effluent
Excess Reclaimed Wastewater
Discharge by Outfall to the Caribbean Sea

FIGURE 4. The flow pattern for water and wastewater on St. Croix with the completion of the bypass.[4]


A WATER REUSE PROGRAM ON ST. CROIX
U.S. VIRGIN ISLANDS
ARTIFICIAL GROUND WATER RECHARGE
OTHER REUSE EXPERIMENTS ON ST. CROIX
ST. CROIX WATER SUPPLY
WATER REUSE PROGRAM

COMPULSORY RE-USE OF WATER DUE TO VERY STRICT WATER POLLUTION CONTROL REGULATIONS IN SALISBURY, RHODESIA

John McKendrick
Chief Chemist, City of Salisbury, Rhodesia

INTRODUCTION

Salisbury, the capital city of Rhodesia, is situated on a wide, high plateau 1 493 metres above sea level, which forms a watershed across the country. The climate is temperate with an average maximum winter temperature of 20°C, an average maximum summer temperature of 27-30°C and an average rainfall of 828 mm.

There are four seasons roughly as follows:-

June to August - Cool, dry season (Winter).
September to November - Hot, dry season (Spring).
December to March - Rains (Summer).
April to May - Post rains, warm (Autumn).

The population of Greater Salisbury is 650 000 and the water consumption varies from 134,05 Mℓ per day during the rainy season to 211,28 Mℓ per day during the hot dry season immediately before the onset of the rains.

The increase in water consumption since 1967/68 is shown in Table 1.

Salisbury lies within the catchment area of its main source of water supply Lake McIlwaine (250 000 Mℓ). Until the construction in 1953 of Lake McIlwaine on the Hunyani River, Salisbury obtained all its water from Prince Edward Dam (3 000 Mℓ) also on the Hunyani River and Cleveland Dam (910 Mℓ) on the Makabusi River which is a tributary of the Hunyani River.

Due to the rapid growth of Salisbury in the early 1970s two new dams have been constructed on the Hunyani River. Henry Hallam Dam (9 200 Mℓ) was constructed above Prince Edward Dam and Lake Robertson (490 000 Mℓ) was constructed below Lake McIlwaine.

The Salisbury urban area drains into the Hunyani River above Lake McIlwaine but below Prince Edward Dam. As a result most of the drainage from the city flows into Lake McIlwaine.

While most of the outer suburban areas of Greater Salisbury have been developed into 0,4 hectare subdivisions and are served by septic tanks, the whole of the central business area, the industrial sites and the higher density residential areas are served by water borne sewerage reticulation.

The increase in total dry weather flow from the two main sewage treatment works since 1955 is shown in Table 2.

The effluent from these works discharged into the Marimba and Makabusi Rivers, which enter the Hunyani River just above Lake McIlwaine.

The quality of the effluent discharged from the sewage works was always kept well within any recognised international standards because it has always been appreciated that high quality effluents were necessary as Rhodesia is faced with the problem of a seasonal rainfall and is not richly endowed with large perennial rivers. Rhodesia has, therefore, to rely on storage dams to maintain an adequate water supply all the year round.

CHANGES IN WATER QUALITY OF LAKE McILWAINE

Water was first drawn from the Lake in November 1953 and until 1959 little change was observed in the quality of the raw water. The quality of water in terms of turbidity, colour and planktonic growth and the ease of treatment of McIlwaine water was similar to that obtained from the other two dams. In common with the Prince Edward and Cleveland Dams a seasonal increase in turbidity during the rainy season entailed an increase in alum dosage, which in the case of Lake McIlwaine amounted to a change from an average 24,0 mg/ℓ to an average 35,0 mg/ℓ. During this period (1954 - 1959) no difficulties were experienced in treating the raw water.

From 1960, periodic algal blooms appeared in the lake and caused purification difficulties at the works. These difficulties, however, were overcome by increasing the alum dosage to 45,0 mg/ℓ and by the use of flocculant aids.

During mid 1964, there was no evidence to suggest that similar problems that were expected to arise later in the year could not be overcome with the same treatment. However, when the temperatures started to rise in the latter half of the hot, dry season, the algae in Lake McIlwaine multiplied at an exceptionally high rate and produced an unexpectedly intense bloom.

This intense algal bloom resulted in severe operational problems at the waterworks such as poor clarification, short filter runs and taste and odour problems in the treated water.

Unfortunately at that time the raw water intake at the lake had fixed draw-off levels which either resulted in de-oxygenated water with a high iron and manganese concentration when the raw water was drawn from the hypolimnium or water with a high algae content which had a very high pH when the raw water was drawn from the epilimnion. This problem was overcome in 1971 by the construction of another intake tower with flexible, telescopic draw-offs. By regular monitoring of the quality of the water at all levels in the lake, this flexible draw-off point can be adjusted so that only the best quality raw water will be sent to the treatment works.

THE CAUSE OF THE PROBLEM

Intensive investigations were made at this time to ascertain the cause of this intensive algal bloom in the lake.

These investigations revealed that the major contributing factor causing this condition in Lake McIlwaine was the drainage from the Salisbury urban area and in particular the sewage effluents which, although of a high quality, contained high concentrations of nitrogen and phosphorus.

This drainage had caused rapid eutrophication of Lake McIlwaine, resulting in a typical eutrophic lake with algal activity confined exclusively to the epilimnion and a reservoir of available nutrients in the hypolimnion. The normal ecology had been disturbed with the prolific development of blue green algae, mainly species of Microcystis and Anabaena.

STUDIES TO REDUCE POLLUTION OF LAKE McILWAINE

The foregoing observations indicated that immediate steps had to be taken to reduce the volume of nutrients entering the lake from the Salisbury urban area if existing conditions in the lake were to be improved.

It was decided to tackle the problem as follows:-

1. Study the physico - chemical removal of nutrients on a laboratory and pilot plant scale.

2. Study the use of sewage effluents for crop irrigation to remove the nutrients from the effluents before they found their way back into the natural watercourses.

PHYSICO - CHEMICAL REMOVAL OF NUTRIENTS

Laboratory studies on the physico - chemical removal of nutrients from raw sewage and sewage effluents were made. As a result of these studies two pilot plants were constructed at one of the sewage works, one to study the physico - chemical treatment of raw sewage and the other to study chemical-biological treatment.

The physico - chemical treatment plant consisted of alum treatment of the incoming raw sewage, followed by clarification, sand filtration, chlorination, activated carbon filters, ammonia removal using clinoptilolite and chlorination of the final effluent.

Analysis of the final effluent from this plant are shown in Table 3.

These results showed that this type of treatment produced an effluent which virtually conformed with the World Health Organisation's standards for drinking water, and showed the possibility of the use of this type of treatment to augment water supplies which may be necessary in the future in this country. However because of the cost involved and the difficulty of importing materials such as activated carbon and clinoptilolite, further studies were discontinued.

The chemical-biological treatment plant consisted of lime treatment of raw sewage followed by clarification, re-carbonation and biological filtration.

Analysis of the effluent from the biological filters are shown in Table 4.

The prime object of this part of the project was phosphorus removal accompanied by good nitrification with minimum amortisation of the existing plant. This has been successfully achieved and further research will be made on this plant with a view to removing or reducing nitrogen and other constituents to acceptable limits.

CROP IRRIGATION OF SEWAGE EFFLUENTS

It has always been appreciated that sewage effluent and sludges are a valuable source of water and nutrients for agricultural purposes.

One of the major sewage works had 160 hectares of arable land and initially a modest programme of land irrigation was embarked upon to study the general removal of nutrients from sewage effluents by land irrigation.

A 5 hectare plot was developed and planted with lucerne. This plot was flood irrigated with sewage effluents under controlled conditions for the purpose of this study. Typical results obtained on the sewage effluent being applied and the regeneration water are shown in Table 5.

Based on the success of these results it was decided to gradually develop the rest of the 160 hectares of land, adjusting the techniques of irrigation and the types of crops as and when necessary.

These studies revealed that flood irrigation was the most economical and that the crops best suited for this type of irrigation were maize, sorghum, sunflower, wheat, lucerne and pasture.

The constant irrigation of these lands with effluent caused springs and streams to run which are normally dry during the dry season. Samples from these and test holes in the lower regions of the lands were constantly monitored and analytical results from a typical sampling point are shown in Table 6.

These results showed that during the dry season the effluent is draining into the soil, giving up its nutrients, mixing with the sub-soil water and thus raising the water table forming springs and streams. In the rainy season, however, the soil is saturated and sewage effluent mixed with rain water tended to run off the surface.

These preliminary studies showed that irrigation of sewage effluents for nutrients removal was successful during the dry season but only partially so during the rainy season.

Although the emphasis was on the primary need to remove nutrients from the effluent, it was realised that this could only be achieved if good farming practices were followed. As a result, land had to be kept dry at certain times of the year so that ploughing, ripening and reaping could be carried out which meant that a large area of expensive land was required.

The rainfall problem and the necessity for dry periods highlighted the difference between this type of farming and that of the normal farmer in that a farmer irrigating crops applies water only when necessary, in fact in this country his main aim is to conserve water, whereas at a sewage works, effluent has to be disposed of for 365 days of the year with the volume being highest during the rainy season due to infiltration and in actual fact this is the time of year when the least amount of water is required for agricultural purposes.

GOVERNMENT REGULATIONS

At about the same time as these studies were being carried out by the staff in the City Engineer's Department, the Rhodesian Government showed its concern about what had happened in Lake McIlwaine and in order to protect the existing and future water resources in Rhodesia, two sets of regulations to control pollution in this country were promulgated. The "Water Pollution Control (Waste and Effluent Water Standards) Regulations, 1971 dealt with the standards of effluents, that may be discharged into streams etc. the main features of which are shown in Table 7.

The "Public Health (Effluent) Regulations, 1972 dealt with the standards required for the re-use of effluents by irrigation, the main features of which are shown in Table 8.

As can be seen from the "Water Pollution Control (Waste and Effluent Water Standards) Regulations 1971 the standards required are perhaps among the severest that exist in the world today. This meant that some form of tertiary treatment of even the highest quality effluents from conventional sewage treatment plants would be required to satisfy these Regulations. Because of this the initial goal of most local authorities and industries was to increase the size of their existing treatment facilities so that an effluent satisfying the Public Health (Effluent) Regulations, i.e. for irrigation purposes, would be produced.

Up until this time the research being carried out by the City Engineer's Department was aimed at reducing the amount of nutrients entering Lake McIlwaine in the hope that this could be done economically and improve the quality of the water in Lake McIlwaine. The advent of these regulations, however, changed the picture completely in that it now became necessary to irrigate all the effluent arising in the City of Salisbury at all times of the year so that the regulations would be complied with.

PRESENT FARMING OPERATIONS

Because of the above regulations the two major sewage works in Salisbury were developed so that they were capable of dealing with the entire water-borne sewage of the Salisbury area. Most of the older sewage works have now been eliminated in order to bring the raw sewage to the two main works.

As previous investigations had shown that irrigation was by far the most economical form of tertiary treatment to satisfy the Water Pollution Control Regulations, a further four farms were purchased by the City of Salisbury for this purpose. The total area of the farming land was now 3 411 hectares, of which 1 397 hectares were irrigable and arable. The main crops grown were maize during the summer months and wheat during the winter months. The problems of the rainy season and of crop farming continued to persist, however, and so further investigations were made using large areas of pasture and forestry.

These studies revealed that pasture irrigation was by far the most successful method of using sewage effluents to remove the nutrients from sewage effluents as the problems of necessary dry periods found in crop irrigation did not exist and that even during the rainy season at least 75% of the nutrients were removed by the surface flow of effluents through the grass.

It was then decided to concentrate on pasture irrigation with cattle running on the farms to act as "mowing machines" to keep the grass under control and also to reduce the costs of pollution control by selling the cattle. Nearly all the irrigable lands were planted with star and kikuyu grasses. The star grass provided rapid coverage, but in most pastures was eventually dominated by the slower growing kikuyu grass. Grass production was more prolific on heavy soils than on sandveld. The pastures are flood irrigated about every three weeks. It has been found that no extra fertilisers have to be used on the heavy soils but that the sandveld required extra fertilisation to increase grass growth and discourage weeds.

Basically three herds of cattle are run on the farms:-

(1) A breeding herd of 900 females using Beefmaster bulls in conjunction with Beefmaster semen.

(2) A breeding herd of 1 000 females being bred to Hereford bulls and Hereford semen.

(3) A breeding herd of 600 which is predominately of the Sussex breed being bred to Sussex bulls and Sussex semen.

At the moment a total of 5 376 head of cattle are run on the farms and although the carrying capacity of the pastures has not reached its full potential it is hoped that 7 000 head of cattle will be run when the pastures are fully developed.

The lush pastures would seem to be a perfect environment for cattle but it has been discovered that these pastures are an ideal incubator for all sorts of pests, diseases etc., and expensive control measures are essential to keep the cattle in good health. Oral dosing is carried out regularly for Liver Fluke and intestinal parasites with vaccinations for Rift Valley Fever, Lumpy Skin, Vibriosis, Contagious Abortion and Enterotoximia. Two recent severe problems encountered have been bloat, especially on pastures grown on the heavier soils, and an infestation of snails causing liver fluke.

The disposal of digested sludge from sewage works has been a recognised problem all over the world. Because of the fear of tapeworm (Taenia Saginata) eggs surviving the mesophilic digestion processes this sludge was never used on the pasture lands. Recent laboratory investigations of the viability of ascaris and taenia saginata ova in the digested sludge revealed that none of the ova recovered were viable. As a result of these investigations certain areas of land are now being irrigated with digested sludge in admixture with final effluent, cattle are being grazed on this land and it is hoped that the incidence of "measly" beef in these cattle will be no higher than that found in cattle which have grazed on pastures irrigated only with final effluent.

Seventy three hectares of trees have been planted to investigate their suitability as an irrigated crop of which eight hectares are poplars whilst the rest are blue gums. The trees have grown well on good soils where they have not been irrigated during the summer, but they became stunted when they were summer irrigated. Two species of blue gums, Eucalyptus Grandis and Eucalyptus Camaldulensis were grown and Eucalyptus Camaldulensis was found to be the most successful. As a result the lands bordering the irrigated pastures are being planted with Eucalyptus Camaldulensis to act as a safety barrier between the pastures and nearby watercourses.

It is interesting to note that the majority of indigenous trees have not been able to stand up to the intensive irrigation and are slowly dying off.

DISPOSAL OF SEWAGE EFFLUENT DURING WET WEATHER

During the long dry season in this country discharges or spillages of sewage effluent seldom occur from a well designed and properly operated disposal system based on conventional treatment followed by land irrigation.

In Rhodesia it is common practice to provide a separate drainage system for the disposal of storm water, and in theory little or no stormwater should enter the sewerage reticulation system during the rainy season. In practice this is not the case, however, stormwater enters the system and considerable increases of flow may occur in reticulation systems from both the infiltration of ground water and the direct entry of surface water. These may be due to one or more of the following:-

(1) Direct ingress of stormwater into sewers through gullies, inspection chambers, manholes or unauthorised connections.

(2) Infiltration at manholes or similar chambers.

(3) High water tables causing sewers and drains to operate under external pressures for which they are not designed.

(4) Ground movement causing cracking of pipes and failure of joints.

(5) Poor standard of pipe laying, allowing infiltration through the joints.

(6) Tree roots penetrating pipe joints.

As a result of this stormwater entering the system, spillages and discharges into natural watercourses can occur at all sections of the disposal system.

These increased sewage flows during the rainy season combined with the problem of run off from saturated irrigation lands caused concern amongst all local authorities in Rhodesia. As the cost of land, equipment etc., to treat all this extra effluent would be substantial, an approach was made to Government asking for assistance or advice to overcome this problem.

Government recognised the problem subsequently produced a policy document "Guidelines for the Disposal of Sewage and Sewage Effluent During Wet Weather", the contents of which are briefly summarised below:-

(1) Exemption permits will not be granted to cover discharges caused by the inadequate capacity of the sewerage reticulation system, pumping installations, or sewage

treatment works, unless:-

(a) the sewer or pumping installations has sufficient hydraulic capacity to accomodate the flows recommended in Table 9.

(b) for a conventional sewage treatment works the various sections of the works are designed with the hydraulic capacities specified in Table 10.

(c) At least one days emergency balancing storage is provided at conventional works.

(d) No discharges will be permitted during the dry season.

(e) Any effluent discharged must have received full secondary treatment and must comply with the following quality standards:-

    (i) Conventional Sewage Treatment Works

    BOD not to exceed 30,0 mg/ℓ.
    Methylene Blue Stability not less than 10 days.

    (ii) Oxidation Ponds

    Dissolved Oxygen not less than 1,0 mg/ℓ.

(f) Where disposal is by means of irrigation on farm lands, the discharge of effluent in excess of 1 x D.W.F. may be permitted provided that:-

    (i) The capacity of the pumping system and the irrigation distribution system is always maintained at not less than 1,5 x D.W.F.

    (ii) Sufficient area of land is provided to dispose of 2 x D.W.F.

RECOVERY OF LAKE McILWAINE

The Hydro-biological Research Unit of the University of Rhodeia has been working in conjunction with the City Engineer's Department to monitor the concentration of nutrients in rivers within the Hunyani catchment area, and in Lake McIlwaine. They have studied the process of eutrophication in Lake McIlwaine and other Rhodesian lakes, comparing this with that in other countries. Methods are being developed to predict the behaviour of water bodies in Rhodesia, and study is being carried out to establish the role of sediments in nutrient storage and exchange. The results of these studies will be used to verify the use of Vollenweider's formula with respect to Rhodesian lakes as conditions in this country are tropical rather than temperate. Of particular importance to the City of Salisbury, the University is engaged in the estimation of a nutrient budget for Lake McIlwaine with special reference to the role of phosphorus. To date these studies have revealed that there has been a considerable reduction in the phosphorus concentrations in the rivers and in the lake itself since the commencement of irrigation of the municipal sewage effluents.

These observations have been confirmed by a reduction of the pH, conductivity, chlorophyll levels, and algal activity in the water in Lake McIlwaine. Secchi disc readings taken by the university staff show that an increase from about 45 cm to about 200 cm has taken place.

All these factors confirm that the lake is in the process of recovery. This is borne out by the fact of easier treatment of the raw water from Lake McIlwaine with the alum dose being used dropping from an average of 55,0 mg/ℓ two years ago to a present day average of 30,0 mg/ℓ. No problems of algae, iron, manganese or tastes and odours are being experienced and as a result a very high quality of treated water is being pumped to the consumers in the City of Salisbury.

SUMMARY AND CONCLUSIONS

1. The unexpected eutrophication of Lake McIlwaine after 1964 caused concern to the City of Salisbury mainly because of the difficulties that arose at the water purification plant.
2. The staff in the City Engineer's Department traced the main cause of this eutrophication to the discharge of treated sewage effluents into the catchment area of Lake McIlwaine.
3. Intensive research was carried out to reduce the quantity of nutrients entering the lake from sewage effluents and it was decided that the most practical and economic method would be by land irrigation.
4. As a result of the Lake McIlwaine investigations fears were expressed that as Rhodesia grew and developed more of her inland lakes would become eutrophic. Rhodesia being land-locked and having a seasonal rainfall, must depend increasingly on storage dams for her water supplies which, unless precautions are taken, could become eutrophic in the same way as Lake McIlwaine.
5. The Rhodesian Government Produced regulations controlling the quality of effluent that may be discharged to natural water courses. These regulations are among the strictest in the world and compelled local authorities and industries to take immediate steps to comply with them. These regulations changed the whole aspect of the water pollution control exercise in that it was now necessary to comply with a set of regulations. Like most local authorities, the City of Salisbury decided to use land irrigation as a form of tertiary treatment.
6. The irrigation of crops proved to be unsatisfactory because of the long periods of dry conditions required therefore a change was made to pasture irrigation.
7. The pollution control and farming operations have been expensive mainly due to the cost of land near the city, but regular monitoring of the regeneration water, streams and Lake McIlwaine has proved it to be successful.
8. The quality of the water in Lake McIlwaine has improved considerably since the land irrigation programme has been implemented.
9. The problems encountered during the wet season have been recognised by Government and steps have been taken to alleviate them.

10. When the nutrient budget for Lake McIlwaine has been estimated by the University of Rhodesia, it is hoped that control measures can be carried out even more effectively and that the present condition of the water in Lake McIlwaine can be maintained or even improved in the future.

ACKNOWLEDGEMENTS

The author thanks Mr. R. Mackenzie, the City Engineer of Salisbury and the Salisbury City Council for permission to present this paper.

REFERENCES

(1) McKENDRICK J. & WILLIAMS R.K. The effects of Urban Drainage on Lake McIlwaine, Rhodesia and Subsequent Water Purification Difficulties. Proc. I.W.P.C. (S.A. Branch) Conf., East London, South Africa. (1968).

(2) McKENDRICK J. The Physico-Chemical Treatment of Raw Sewage. M.Phil Thesis, University of Rhodesia. (1973).

(3) NEELY A.B. Nutrient Removal in Sewage Treatment. M.Phil Thesis, University of Rhodesia. (1975).

(4) McKENDRICK J. Water Pollution and Its Control In Salisbury. Proc. Rhodesian Science Congress, Salisbury, Rhodesia. (1973).

(5) Guidelines For The Disposal of Sewage and Sewage Effluents During Wet Weather. Ministry of Water Development, Rhodesia.

TABLES

TABLE 1

VOLUME OF WATER PUMPED TO CITY (Mℓ)

Year	Volume Pumped (Mℓ)	Year	Volume Pumped (Mℓ)
1967/1968	34 083,3	1973/1974	44 298,8
1968/1969	35 771,4	1974/1975	55 023,3
1969/1970	39 069,2	1975/1976	56 898,2
1970/1971	43 073,6	1976/1977	57 654,2
1971/1972	43 322,6	1977/1978	60 016,0
1972/1973	50 598,0		

TABLE 2

DRY WEATHER SEWAGE FLOWS

Year	D.W.F. (m^3) per day	Year	D.W.F. (m^3) per day
1955	19 090	1967	39 545
1956	17 954	1968	38 045
1957	18 182	1969	40 045
1958	25 454	1970	40 182
1959	26 363	1971	46 136
1960	28 182	1972	52 136
1961	29 091	1973	57 000
1962	33 636	1974	70 097
1963	32 727	1975	79 607
1964	35 454	1976	81 088
1965	34 545	1977	76 481
1966	37 273	1978	86 295

TABLE 3

pH	- 6,8	Arsenic (mg/ℓ As)	- NIL
Colour (Hazen Units)	- 5	Chloride (mg/ℓ Cℓ)	- 57,0
Dissolved Oxygen (mg/ℓ)	- 1,8	Total Chromium (mg/ℓ Cr)	- NIL
COD (mg/ℓ)	- NIL	Detergents (mg/ℓ Manoxol O.T.)	- 0,05
4 Hour Permanganate Value (mg/ℓ)	- 0,8	Fluoride (mg/ℓF)	- 0,15
Suspended Solids (mg/ℓ)	- 1,2	Iron (mg/ℓ Fe)	- 0,08
Total Dissolved Solids (mg/ℓ)	- 428,0	Manganese (mg/ℓ Mn)	- 0,08
Soap, Oil, Grease etc. (mg/ℓ)	- NIL	Total Nitrogen (mg/ℓ N)	- 0,3
Free and Saline Ammonia (mg/ℓN)	- 0,1	Total Phosphate (mg/ℓ P)	- 0,018
		Sulphate (mg/ℓ SO_4)	- 222,9
		B. Coli (No. per 100 mℓ)	- NIL

TABLE 4

pH	- 8,0	% Methylene Blue Stability	- 100%
Settleable Solids (cm^3/ℓ)	- 0,8	Total Alkalinity (mg/ℓ Ca CO_3)	- 130,0
Suspended Solids (mg/ℓ)	- 70,0	COD (mg/ℓ)	- 37,5
4 Hour Permanganate Value (mg/ℓ)	- 5,5	Dissolved Oxygen (mg/ℓ)	- 5,9
Chloride (mg/ℓ Cl)	- 48,0	Colour (Hazen Units)	- 15
Free and Saline Ammonia (mg/ℓ N)	- 2,5	Turbidity (Jackson Units)	- 2,8
Nitrite (mg/ℓ N)	- 0,45	Total Phosphate (mg/ℓ P)	- 0,6
Nitrate (mg/ℓ N)	- 24,0		

TABLE 5

	Humus Tank Effluent Before Irrigation of Lucerne	Regeneration Water below Lucerne Plot
Total Phosphate (mg/ℓ P)	8 - 10	0,2
Free and Saline Ammonia (mg/ℓ N)	9 - 11	TRACE
Nitrate (mg/ℓ N)	4 - 6	20-25*

* The increase in nitrate is understandable as the crop was lucerne.

TABLE 6

Month	o-Phosphate (mg/ℓ P)	Total Phosphate (mg/ℓ P)	Free and Saline Ammonia (mg/ℓ N)	Nitrate (mg/ℓ N)	Chloride (mg/ℓ)
June	0,01	0,07	-	Trace	18,0
August	0,13	0,93	0,8	Nil	25,0
September	0,1	0,26	1,3	Nil	20,0
November	2,0	2,71	4,5	Nil	102,0
December	2,5	3,73	2,0	2,5	62,0
Typical Sewage Effluent	9,0	9,3	12,0	14,0	180,0

TABLE 7

THE WATER POLLUTION CONTROL (WASTE AND EFFLUENT WATER STANDARDS) REGULATIONS 1971

pH	- 6,0-9,0	Arsenic	- 0,05 mg/ℓ As
Temperature	- 35°C	Chlorides	- 100,0 mg/ℓ
Dissolved Oxygen	- 60% Saturation	Chromium	- 0,05 mg/ℓ Cr
		Detergents	- 1,0 mg/ℓ Manoxol O.T.
COD	- 60,0 mg/ℓ		
Oxygen Adsorbed	- 10,0 mg/ℓ	Fluoride	- 1,0 mg/ℓ F
Suspended Solids	- 25,0 mg/ℓ	Iron	- 0,2 mg/ℓ Fe
T.D.S.	- 500 mg/ℓ	Manganese	- 0,1 mg/ℓ Mn
		Total Nitrogen	- 10,0 mg/ℓ N
Soap, Oil, Grease etc.	- 2,5 mg/ℓ	Total Phosphates	- 1,0 mg/ℓ P
		Sulphate	- 200 mg/ℓ SO_4
Free & Saline Ammonia	- 0,5 mg/ℓ N	Zinc	- 1,0 mg/ℓ Zn

TABLE 8

PUBLIC HEALTH (EFFLUENT) REGULATIONS, 1972

EFFLUENT FROM SEWAGE TREATMENT WORKS

	Type of usage or crops	Minimum standards of purity of effluent	Method of irrigation	Other requirements
A.(a)	Grain crops; and	(1) Biochemical oxygen demand not exceeding 70 parts per million; and		
(b)	Crops grown for industrial processing such as oil-seeds, fibre etc. which are not for direct human consumption, but excluding crops grown for dehydration, canning or preserving; and	(2) Stability as measured by the methylene blue test not less than 36 hours.	Surface only	
(c)	Crops grown solely for seed-production for sale to registered seed merchants but not human consumption; and			
(d)	Nursery production, excluding cut flowers grown for sale; and			
(e)	Fodder crops for harvesting; and			
(f)	Pastures for slaughter stock; and			No grazing to be permitted within 24 hours of application of effluent, and drinking troughs of potable water to be provided for stock.
(g)	Deciduous and citrus orchards, trellised vines, plantation and tree crops.			No fruit windfalls to be marketed.
B.	As in A(a),(b),(c),(d),(e) and (f)	(1) Biochemical oxygen demand not exceeding 30 parts per million; and		

Type of usage or crop	Minimum standards of purity of effluent	Method of Irrigation	Other requirements
	(2) Stability as measured by the methylene blue test not less than 10 days.	Surface or sprinkler	As for A
C.(a) As in A; and (b) Pastures for dairy stock; and (c) Cut flowers grown for sale.	(1) Biochemical oxygen demand not exceeding 10 parts per million; and (2) Stability as measured by the methylene blue test not less than 21 days; and (3) E. Coli (type 1) not exceeding 1000 per 100 millilitres.	Surface or sprinkler	As for A(f) and (g)
D. Public amenities, e.g. sports fields, public parks, golf courses, etc. but not swimming pool surrounds.	(1) Biochemical oxygen demand not exceeding 10 parts per million; and (2) Stability as measured by the methylene blue test not less than 21 days; and (3) E.Coli (type 1) not exceeding 1000 per 100 millilitres; and (4) Residual chlorine not less than 0,3 parts per million after 30 minutes' contact in samples taken at the sewage-treatment works.	Surface or sprinkler	

TABLE 9

Dry Weather Flow (1 000 m^3/day)	Minimum Design Factor for sewer Capacity (Multiply of D.W.F.)
0 - 2	5,25
2 - 6	4,50
6 - 20	3,75
20 - 200	3,00
200 or more	2,70

TABLE 10

Treatment	Minimum Design Capacity
At least screening	All Flows
At least screening and primary settlement	5 x D.W.F.
Full secondary Treatment	3 x D.W.F.

INSTITUTIONAL FACTORS

AN ASSESSMENT OF LEGAL AND INSTITUTIONAL BARRIERS TO PLANNED REUSE OF WATER IN THE COLORADO RIVER BASIN

J. Gordon Milliken
Senior Research Economist
Denver Research Institute
University of Denver
Denver, Colorado 80208

and

Loretta C. Lohman
Research Associate
Denver Research Institute
University of Denver
Denver, Colorado 80208

Introduction

This paper deals with the policy aspects of planned water reuse in the 242,000 square mile area of the Colorado River Basin. The paper does not discuss water reuse technology nor systems of water treatment that would make reuse possible, but instead focuses on reuse policy, i.e., the system of control, planning and management of water resources in a region that faces serious problems because of the inadequacy of its water resources.

For several reasons, the water resources of the Colorado River Basin are inadequate in quantity to meet all legitimate demands for their use, even though water is customarily utilized by a succession of users as it flows downstream. The growing demands for water by metropolitan populations, by manufacturing industries, and by mineral and energy developers, in addition to the heavy dependence on water by agriculture, will cause an ever-growing gap between water supply and need. This will force both a curtailment of some demands and a reallocation of water supplies among potential users, resulting in some negative economic, social and environmental consequences.

A companion problem is that of water quality. The Colorado River grows naturally salty as it flows southwest, and nearly all users add to its natural salinity. In the lower basin states (Arizona, Nevada and California), salinity reaches levels that reduce the river's usefulness and cause economic penalties to many water users. As expected economic development continues, salinity will increase unless offset by salinity control measures which are often difficult and expensive.

This paper focuses on *planned reuse* of water in the Colorado River Basin, which we distinguish from repetitive use of water by a succession of water rights holders along a stream system. We define planned reuse as: A deliberate second or repetitive use of water by the same or another user, with or without treatment, after either: (1) direct transfer of the used water from one use (or user) to the next, so that the water is not returned to the alluvial stream system for reallocation according to the existing water rights doctrine; or (2) groundwater recharge of used water (normally treated wastewater) under controlled conditions for delivery by the aquifer to groundwater users in a specific region.

Planned reuse appears likely to help alleviate the water problems of the Colorado River Basin in two ways:

- planned reuse can, in the proper circumstances, help achieve greater utilization of existing water resources without increasing degradation of water quality; for example, planned reuse may result in lower evaporation and phreatophyte losses

eceding page blank

than an equal number of sequential uses and returns to the stream system;

- planned reuse can improve water quality in the Colorado River system if known supplies of low quality water are selected for uses that are both consumptive and adaptable to low quality water. The resulting consumption will, of course, remove a source of quality degradation and thus improve the quality of water remaining in the river system. (The same benefit will come from planned first use of naturally low quality water, as from planned reuse of water which has been degraded in quality, so long as the use is consumptive.)

From a policy viewpoint, achieving a greater degree of planned reuse requires: (a) identifying and overcoming various legal, institutional, political and economic obstacles to reuse, and (b) adopting strategies to promote reuse, by reinforcing existing incentives and facilitating factors or by creating new incentives. This paper attempts to show how to do both.

CURRENT AND POTENTIAL TYPES OF PLANNED WATER REUSE

Unplanned water reuse by a succession of water rights holders is a regular occurrence in the Colorado River Basin. Specific, planned reuse is a less frequent occurrence, although stimuli for it are increasing in rough proportion to population and industrial growth in an area of limited water resources. Nearly all current and most contemplated reuses are non-potable applications.

A common type of reuse of wastewater, usually municipal sewage effluent, is for turf irrigation for recreational purposes, such as golf courses, parks and landscape areas around public buildings. Municipal effluent (and in a few cases industrial wastewater) is used for irrigation of certain agricultural crops, either through a water exchange program between an irrigation district and a municipality, or as part of the land application method of advanced wastewater treatment. Aquaculture--growing fish or vegetable products in a controlled water environment--is another possible use of effluent which can add to the food supply and provide advanced wastewater treatment. Irrigation reuses have the advantage of making use of many of the nutrients present in previously used water, as well as reducing some treatment costs if the wastewater is not laden with toxic or disease-bearing organisms. Municipal wastewater usually is given at least secondary treatment before reuse for irrigation, and there are some applications for which greater treatment is required by health regulations, as discussed below.

Industrial reuse of water is becoming more common with the increasing costs of permissible means of effluent discharge, e.g., controlled evaporation, and with current and projected increases in costs of buying and transporting fresh water. In-plant recycling of industrial cooling or process water, with whatever treatment is required, can occur through several cycles until the water is no longer usable. This approach not only saves on initial water costs, but also reduces costs of evaporation pond disposal caused by zero-discharge requirements. Furthermore, it reduces the volume of stream depletion resulting from consumptive disposal where once-through water use is practiced. A number of industries, most often in California, have taken this in-plant approach for processes ranging from canning to steel production. However, complete recycling is most often found in power generating plants. Frequently wastewater provides the intake water supply for power plants, resulting in additional reuse.

Industries which can and have reused water, through both in-plant recycling and intake of used water, include food and leather products, primary metals, petroleum and coal operations, chemical production, textile, lumber and paper production, and mining operations. Coal slurry pipelines, depending upon the location, might also use wastewater as a liquid medium. Furthermore, pipeline transport water is generally reused for cooling at the pipeline terminus, resulting in still another form of reuse.

Groundwater, or aquifer, recharge is an area of water reuse which is develo[illegible] appeal in areas where groundwater has been mined. Among the large recharge projects are two in southern California (Central and West Basin Replenishment District and Orange County Water Factory 21) funded by local agencies, and an experimental program in Phoenix which is operated by the U.S. Department of Agriculture Water Conservation Laboratory.

A final category of reuse and potential reuse is the creation of recreational lakes and wildlife habitat through the use of wastewater. Several lakes have been established, most prominently in California and Texas, and have met some success in terms of public acceptance and in meeting safety standards. Habitat, such as that created by effluent flow in Las Vegas Wash and in the Phoenix area's Salt and Gila River greenbelts, is being maintained and created with wastewater.

Quality Constraints On Reuse

Salinity (TDS) concentrations in wastewater can be as high as 35,000 mg/l (seawater). A general rule of thumb is that water up to 3,000 mg/l is usable by industry and for recreation and wildlife habitat, although certain industries might have to resort to some form of demineralization. Saline water with TDS ranges as high as seawater has been successfully used for irrigation of salt-tolerant strains of grain and vegetable crops in Israel, Australia, and in the U.S. by scientists of the University of California, Davis. Recreational lakes and habitats can tolerate varying degrees of salinity, depending on the intended use. Only in the case of recharge of groundwater, which may be withdrawn after mixing for possible potable use, might the drinking water standards of 500 mg/l TDS be applied at some step in the recharge process. Of course, these TDS standards, along with a full range of safety standards, would be applied for any potable water reuse.

Most other quality factors handled by effluent treatment (BOD, fecal coliform, etc.) do not impact on industrial uses except for food product manufacture. Depending on the type of reuse, treatment may be required when heavy metals, nitrates, phosphates and certain other contaminants are present. Contaminants such as radioactivity, toxic chemicals, pesticides or viruses are of concern in groundwater recharge, or in any area where human body contact or food supply might be affected. Most of these elements are readily treated, if not by secondary, then by advanced wastewater treatment.

Quantity Constraints On Reuse

The most difficult factor in determining the effect of a particular reuse is the amount of water consumed in relation to the quality or quantity of water diverted. In industrial processes where water is recycled, the diversion of fresh water can drop to 10-12 percent of total water needs. On the other hand, irrigation with saline water can require about four times the normal water application. Other reuses might not change total water consumption but might improve instream water quality. And, when the intake is of low quality, the capability to recycle might be reduced thus requiring larger amounts of makeup water. For instance, power plants can recycle low quality water fewer times, thus requiring greater diversions to provide sufficient cooling water.

LEGAL BARRIERS TO PLANNED REUSE

Barriers Found In Water Allocation Law

The Colorado River is controlled and apportioned for use by a complex system of international treaties, federal legislation, interstate compacts, state water laws, and contracts. These have been collectively described as the *Law of the River*. These fundamental water control patterns contain some serious barriers to planned reuse, but also some reuse incentives.

Compact Limitations On Consumptive Use. The Colorado River Compact and supplementary interstate agreements that allocate Colorado River water among the states of the CRB provide an ultimate limit on the consumptive use of river water which can be made in each of the states. This limitation would not necessarily affect the type of consumptive use to which a state's apportionment is devoted, but the states would be individually forced to choose among water-consuming alternatives. If certain proposed planned reuse projects would result in greater consumptive use of water, it is an intrastate decision whether or not to proceed with such projects, weighing their merits (e.g., in improved water quality) against the increased consumptive use that results. Until the states of the CRB approach the limits of consumptive use allocated to each by interstate agreements, there will not be a serious constraint on reuse plans that increase consumption more than first uses of water do.

The Colorado River Compact (and the interstate allocations of water that supplement the compact) is considered fundamental to continued regional cooperation among the seven CRB states, which in turn enables the region to promote the water resource development projects necessary for its continued economic development. It is theoretically possible to visualize new interstate agreements that would modify allocation commitments to permit planned reuse projects that would result in more efficient CRB water uses, but any such agreements that threatened the stability of the compact or its supplementary allocations would be strenuously opposed by the water resource authorities of the entire basin. When the limits of consumptive use of CRB water allocated to each state are approached, it is likely that the interstate tensions over the compact will become much more serious than at present and may result in some adjustments. Now and for some years to come, however, there is no apparent benefit great enough to justify the risk of tampering with the compact, so where it presents a barrier to planned reuse, it will continue to do so for the foreseeable future.

Implied Compact Preference For Domestic And Agricultural Use. As with the previous barrier, this is a potential future obstacle to planned reuse rather than a current constraint. An arguable interpretation of the Colorado River Compact leads to the position that, once the Upper Basin develops to the point that deliveries to the Lower Basin must be reduced, and this reduction threatens to curtail Lower Basin domestic and agricultural uses, the Upper Basin must instead curtail its other uses, e.g., industrial use, power generation, and energy development, because of a compact preference for domestic and agricultural uses. This circumstance has not occurred, and until the threat becomes imminent or seriously deters development of Upper Basin energy projects, there will be little support for action to reinterpret or modify the compact to resolve the question.

Federal And Indian Reserved Water Rights. The so-called reservation doctrine states that if the United States, by treaty, act of Congress or Executive Order reserves a portion of the public domain for a federal purpose which will ultimately require water, and if at the same time the government intends to reserve unappropriated water for that purpose, then sufficient water to fulfill that sole purpose is reserved, with the land, from appropriation by private water users. Governmental intent to reserve the water is inferred if the previously unappropriated waters are necessary for the accomplishment of the primary purpose for which the reservation was created.

Under the reservation doctrine, the federal government has a great deal of flexibility in its exercise of water rights. The strengths of the federal rights are twofold: (1) when reserved water is eventually put to use, the right of the United States is superior to private water rights in the same water source which were initiated after the date of the federal reservation, hence subsequent private rights may be impaired or destroyed without compensation through the exercise of the reserved right; and (2) the federal use of reserved water is not subject to state laws regulating the appropriation, use, or forfeiture after nonuse of water within the state, although the United States can be joined in any suits for the adjudication or administration of water rights in a water source, under the McCarran Amendment.[1] Indian water rights are a form of reserved rights. The amounts of water reserved for Indian rights are generally agreed to be measured by the amount of water required for irrigation of all lands within the Indian reservation that are susceptible of irrigation. There is lingering uncertainty over the ownership of water rights allocated under state law but threatened with future preemption, without compensation, by the federal government for its use or use on Indian reservations. This uncertainty may already, and certainly can potentially, inhibit investment in water development projects by individuals, firms or local governments. In the case of investments by private firms in industrial and energy development projects dependent on state-allocated water rights, the inhibition is likely to constrain planned reuse because such projects have a high potential for reuse applications.

Several potential solutions to the reserved water rights dilemma can be visualized, although none is fully satisfactory to all parties: (1) the owner(s) of state water rights can continue to sue to adjudicate or quantify the federal reserved right, under the McCarran Amendment; (2) the owner of the state water right can proceed to develop the water-using project, setting aside financial reserves as a contingency fund to obtain other rights (or to lease the existing right from the federal government or an Indian tribe) in case of successful preemption; (3) congressional legislation may be sought, whereby the federal government assumes the cost of litigation to adjudicate or quantify any federal reserved rights on a water source for which a state government serves notice of a pending rights conflict; (4) legislation may be sought in the Congress whereby the federal government would indemnify holders of state water rights in case of preemption, particularly in cases where the state water rights holder develops a project of "national interest" (e.g., energy resource development); (5) a variation of the last proposal would involve federal provision of low-interest "disaster" loans to holders of state water rights that are preempted or (6) congressional legislation may be sought to have the federal government abandon its reserved water rights and obtain rights, when needed, through purchase or condemnation.

The legal complexities and uncertainties surrounding federal and Indian reserved water rights are too involved for more than a summary analysis in this paper, particularly since the topic impacts all water uses, not only planned reuse. However, there appears to be strong support among state water officials and most water users in the Colorado River Basin (presumably excluding the Indian tribes) for a solution to the reserved water rights dilemma that does not continue to place owners of state water rights in an indefinite position of heavy economic risk. The continued tolerance of the existence of such risk may reflect a general belief that widespread preemption of state water rights under the reservation doctrine is so intolerable and inequitable an outcome as to be unenforceable or to require federal action in the form of economic relief.

Of the six potential solutions outlined above, numbers (1) and (2) are already available to individual water rights holders, but are expensive, uncertain as to outcome, and provide only localized solutions that do not solve the regional reserved rights dilemma. The remaining four potential solutions place responsibility for initiating a solution on the federal government and also place there some or all of the financial burden of the solution. From the viewpoint of western state governments and most water rights users, this is the proper and equitable place for the responsibility to lie. A contrary view is probably held by federal officials, Indian tribes and their supporters, and by officials from states that are not impacted by the reserved rights doctrine, e.g., eastern and southern states with few federal reservations and ample water supplies. The feasibility of having the federal government assume responsibility for the solution is unclear but worthy of continued attention by CRB state officials.

State Water Law Restrictions On Changes In Rights. CRB states' statutory restrictions on changes in water rights in terms of point of diversion or use, or purpose of use, apply equally to all water right changes. However, they affect reuse especially because most planned reuses would require changes from the historical exercise of the water right concerned. The statutory procedures for obtaining approval of changes in water rights require several actions, reviews or hearings, and possible appeals, all time-consuming. Besides the inherent delays, the procedures pose two major difficulties to potential planned reuses. First, the legal costs of preparing and pursuing an application for a change of water right can be significant. Second, there is substantial uncertainty as to the extent of the water right that will result from the proceeding, until a determination of historic consumptive use is made. Where the statutory restrictions on changes in water rights present a barrier to a specific reuse application, it seems appropriate to seek a tactical solution specifically tailored to that reuse plan. Such a solution could result through negotiating for a water right that can accommodate the planned reuse with least interference with the rights of other users, or by altering the technical aspects of the planned reuse to minimize the need for radical changes in point of diversion or use, or by otherwise compensating other water users for the damage caused by such changes. Generally, the state restrictions on changes in water rights are reuse barriers that should not be overcome through fundamental or radical alteration of state water law.

State Water Law Restrictions On Changes In Stream Regimen And State Water Law Restrictions On Water Quality Degradation. As is true of the previous barrier, state water laws within the CRB contain restrictions on actions by water users, designed to avert injury to other water users. These restrictions constrain actions that alter the regimen of the stream, e.g., the rate and timing of flow, and that unreasonably degrade the quality of water returned to the stream for use by others. While both types of restrictions apply to all uses of water, they represent a particular barrier to planned reuse. Recapture of return flow and its reuse may well alter the regimen of the stream. Reuse may add impurities to water beyond those added by a single use, and may also increase consumption thus concentrating existing impurities. These consequences may be restrained by state water rights or water quality administrators, or more commonly enjoined by the injured party through legal proceedings. It should be noted, however, that planned reuse often may occur despite the existence of legal barriers, since in many cases the change in stream regimen or in water quality degradation is so minor as to cause no significant injury to downstream water users. Even when it does, the potential reuser has available options: modifying his plans to minimize injury; purchasing other water rights that compensate downstream users for the injury caused by reuse; or by paying indemnification for the damages caused. In summary, the barriers to planned reuse caused by state water law restrictions are not unreasonable ones, and it would be unwise to amend them, since the damage to a long-entrenched and accepted system of water rights discipline would far outweigh any apparent benefit gained by stimulating planned reuse.

Colorado's Prohibition Of Recapture And Reuse Of Water. Unique among CRB states, Colorado water law specifically prohibits a water user from recapturing return flow, after first use, for subsequent reuse. This prohibition does not, however, represent an absolute barrier to planned reuse. *First,* foreign water (i.e., water imported from another stream system) is exempt and may be freely reused. This exemption is the basis for the ambitious plans of the Denver Water Board to recycle municipal wastewater for domestic--ultimately potable--use. Only that portion of wastewater evolving from transbasin diversions from the CRB would be reused, the portion originating in Denver's South Platte River Basin would be released after a single use. *Second,* Colorado law does not interpret closed system recycling, e.g., recycling of power plant cooling water, as recapture and reuse of return flow but instead treats this as a type of consumptive use. Thus an industrial reuser may purchase an existing water right and reuse the water to ultimate consumption, provided the historic consumptive use is not exceeded and the stream regimen is maintained. *Third,* since the purpose of the prohibition on recapture of return flow is the protection of downstream water users, Colorado law does permit a water rights holder to recapture and reuse, provided all downstream users are compensated through provision of a supplementary water supply from another source or possibly by payment of monetary damage compensation. In these circumstances, Colorado's prohibition of recapture and reuse does not appear to impose an unreasonable barrier to planned reuse, and an amendment to Colorado law to encourage reuse would cause an unwarranted and unwise disruption to Colorado's water rights system.

OTHER LEGAL AND REGULATORY MEASURES AFFECTING PLANNED REUSE

Several federal and state legal and regulatory measures, other than those concerned with water allocation, also directly or indirectly affect planned reuse. These include both incentives and barriers to reuse.

Federal And State Water Quality Control Laws, Including And Supplementing The Federal Water Pollution Control Act. The pervasive and increasingly stringent federal and state programs for water quality control are not barriers to planned reuse; indeed, they serve as a major incentive to planned reuse in the CRB. Industrial and municipal water users face increasingly restrictive and expensive treatment standards before used water can be returned to the stream system, and for many water users the least expensive alternative is to cease wastewater discharges completely. For municipalities, this often may take the form of (tertiary) land application treatment of secondary-treated effluent--a form of agricultural reuse. Alternatively, it may take the form of conveying treated effluent to an industrial reuser or to independent agricultural reusers. For industrial reusers, the constraints of water quality control programs encourage reuse so long as the water remains of adequate quality for use or can be economically treated to restore quality between uses. Ultimately, quality degradation requires disposal of the reused water. In rare cases, industrial effluent can be further reused beneficially without constituting a threat to stream quality. In most cases, however, the effluent must either be treated to remove pollutants (which is usually the more expensive alternative) or evaporated to complete consumption in contained ponds. When disposal by evaporation is chosen, the industrial reuser normally will minimize his costs by maximizing the number of reuse cycles of intake water, thus achieving the smallest volume of effluent requiring evaporation. This economic incentive also helps achieve conservation of water resources in the CRB.

Federal And State Environmental Statutes Indirectly Inhibit Reuse. Several statutes intended to protect the natural environment in some way have a secondary effect that either directly interferes with a planned water reuse project or adds delay, expense or risk that inhibits such a project. Such statutes include those that prohibit diversion of water needed for a reuse project (e.g., Fish and Wildlife Coordination Act, Endangered and Threatened Species Preservation Act); or prohibit siting of an industrial or municipal facility that would reuse water (e.g., Wild and Scenic Rivers Act, Clean Air Act, and various state laws affecting power plant siting in specific areas, either to preserve desired natural features or to avoid environmental or public safety hazards); or which require extensive research studies prior to permit approval (e.g., National Environmental Policy Act and similar state environmental protection, facility permit and land use statutes). These statutes may inhibit a water reusing project because of inherent natural conflicts; e.g., an otherwise desirable reuse facility may have been unwisely planned in an area where it would cause severe air degradation or incur a high risk of seismic failure. In other cases, however, the statutes may block a desirable project for a relatively trivial infringement of statutory authority. The environmental statutes do provide a weapon to political opponents of various development projects, since the statutes favor preservation of the status quo and place the burden of proof on proponents of any development action that alters the existing environment. Used as a strategy by advocates opposed to development, civil litigation based on alleged violation of environmental statutes has proven extremely effective in causing extensive delays in development schedules, major increases in construction costs, and some ultimate denials of operating permits. It is apparent that the consequences of such litigation have in some cases gone beyond the intent of the authors of the statutes.

Overcoming the barriers to planned water reuse represented by environmental legislation is difficult although rarely impossible. There seems no likelihood of repeal or major amendment of these statutes since they retain much popular support and generally are seen as promoting the public interest. Excessive abuse of certain statutes, e.g., the Endangered and Threatened Species Preservation Act, can lead to amendments that permit reasonable exceptions or exemptions from overly restrictive statutory provisions. Where patterns of abuse can be shown to frustrate the will of the majority most directly affected, it seems possible that statutory amendments might be passed permitting appeal to an interagency review committee or permitting exceptions authorized by a state governor when certain conditions have been met. Gubernatorial exception may be the best way to overcome barriers that are unduly restrictive to water development projects in the semi-arid West, but which present no serious problems in other regions having ample water resources.

Even where no statutory amendment is possible or desirable, barriers often can be overcome through more enlightened tactics by development advocates. Many of the more serious frustrations faced by development proponents in the early 1970's occurred either because the planned development would have caused grave damage to the environment or unnecessary risk to the public, or because the proponents neglected to do the planning or research necessary to adequately state their case. By 1979, development advocates appear to have more experience and sophistication in understanding the intricacies of environmental legislation. They do not attempt to obtain permits for hopeless cases, and when they propose a controversial project involving power generation or dam construction they are likely to have researched impacts, studied alternatives and planned mitigation. For projects which involve planned water reuse and which result in improved stream quality and/or greater beneficial use of scarce water resources, emphasis on these favorable environmental consequences is likely to promote approval even when some adverse environmental effects exist.

INSTITUTIONAL BARRIERS TO REUSE

Various institutional relationships and forces interact to create a system of management for water resources in the Colorado River Basin. Some of these relationships and forces present barriers to planned reuse, while others promote reuse.

Conflicting Institutional Pressures Constrain CRB Water Management Decisions. Decisions by state officials on water management in the Colorado River Basin are significantly constrained by numerous conflicting pressures stemming from interstate agreements on water allocation, intrastate development plans, federal policies on water, environmental control, and energy development, etc. These pressures tend to inhibit innovations in water management and force water managers into decisions and policies based on precedent and tradition. Although there are no identifiable groups opposed to planned reuse as such, there are likely to be frequent instances in which a planned reuse potentially threatens one of the existing interests or conflicts with an existing policy. The strong pressures on state officials who allocate water or administer water quality have reduced their flexibility and discretion in decision-making and have led to narrow interpretations of statutes and regulations affecting water use. For example, a proposal for land application of municipal effluent (for tertiary treatment) would likely be favored by municipal officials and by EPA, opposed by downstream water rights holders and perhaps opposed by state public health officials. In this situation, the water allocation agency that must decide on questions of historic consumption and river regimen is likely to decide according to precedent rather than to propose innovative alternatives that could facilitate reuse. No solution is apparent that would lessen the complex interplay of interests in the CRB and thus remove the disincentive to innovation.

Varying State Water Policies And Definitions of "Public Interest" May Inhibit Reuse. Differences in economic development status and in natural resources underlie differences among CRB states in their policies on water use (e.g., energy and industrial development vs. agriculture). Such policies may not take precedence over the existing legal system of water allocation, but when they enjoy popular support and are voiced by a governor, they can strongly influence the discretionary decisions of state water allocators. They also can be reflected in state statutes, as evidenced by statutory barriers to export of water in coal slurry pipelines. Where such policies restrict transfers of agricultural water to energy, or to export of any water in coal slurry, they can effectively bar these planned reuses. These will remain as institutional barriers, not subject to solution, unless circumstances change so that planned reuse also becomes a recognized matter of public interest and a priority of state water policy.

Conflicts Between Water Allocation And Water Quality Interests Can Inhibit Reuse. Conflicts between water allocation and quality agencies occur both at federal and state levels and throughout the CRB, even where the agencies are combined in a single organization. These conflicts can inhibit planned reuse in several ways. In most CRB states, water allocators prefer and encourage return of used water, even when it may marginally exceed NPDES standards, rather than incur substantial consumptive losses through evaporation when treatment is uneconomic. This position is maintained during water rights transfer proceedings by a rigid interpretation of "historic consumptive use" which places an economic penalty on users who reuse intake water and evaporate effluent rather than return it after a single use. Water allocators also generally disfavor land application of municipal wastewater. Water quality officials inhibit planned reuse in other ways.

They are likely to discourage any reuse that may increase the likelihood of human contact, including irrigation reuse of municipal effluent on certain crops, or groundwater recharge that may directly impact a municipal water supply. Water quality officials may also inhibit certain water uses or reuses through an overly strict interpretation of water quality control regulations.

Although the interests of water allocation and water quality agencies have inherent and fundamental conflicts, there is an opportunity to find more areas of compromise through better interagency cooperation and by increased recognition that more planned reuse of water can help solve both water quality problems and some water quantity problems.

SOCIETAL ATTITUDES THAT ARE REUSE BARRIERS

Public Attitudes To Reuse Are Generally Favorable But Influenced By Professionals. To the extent that the general public has considered planned reuse of water, the concept is generally favored. This is particularly true in semi-arid areas such as the CRB where the need to conserve water and to optimize its beneficial use is widely accepted. There is greater resistance to proposals involving direct human contact with reused water, particularly for potable use, from a combination of aesthetic distaste and health concerns. Even there, however, research to date indicates that persons are likely to accept such use once professional water managers and public health officials recommend it. Public acceptance, in fact, exceeds the degree of acceptance by professionals. The latter will be convinced only after adequate research shows that direct contact recycling techniques are perfected to the point that recycled water is fully as safe as water from conventional sources. This appears to be a necessary and desirable barrier, which will be overcome by additional research and development in water reuse technology. Non-potable reuse does not face any significant barrier from public opinion, although some health and safety objections are raised by professionals over certain reuse applications. The public appears to favor non-potable reuses, particularly those which result in aesthetic benefits.

Conflicts Exist Among Parties-At-Interest Over Development Requiring Water Use Or Reuse. Reuse plans that involve facility construction or economic development often become embroiled in conflicts, not over the reuse itself, but over other economic, environmental, or social consequences of the development plan. These societal attitudes or positions often are strong and pervasive, and often overwhelm attitudes favorable to the concept of reuse itself. Where these barriers exist, there is little that can be done until the conflict is resolved, except to assure that the benefits of planned reuse are made visible to the contending parties.

BARRIERS AFFECTING WATER REUSERS

The primary impetus for choice of a planned reuse option is economic, thus the fundamental barrier affecting potential reusers is that reuse is frequently more expensive than alternatives. There also are technological barriers to some planned reuse applications.

Planned Reuse Results In Higher Costs To The Reuser. Planned reuse of water for industrial processes often involves costs to the reuser as undesirable substances increase in concentration and result in corrosion, scaling, and more frequent equipment repair or replacement. Treatment of the water during reuse, to avoid such problems, also causes costs to the reuser. Whenever the costs of reuse exceed the costs of conventional one-time use of water (including costs of pretreatment, post-treatment, and disposal), reuse would not take place. The only exceptions arise when there are political or public relations benefits to reuse (e.g., in furthering public acceptance, or increasing the likelihood of state permit approval for a facility that reuses water). Financial incentives also could be provided to potential reusers that would make reuse attractive; analysis of such incentives is the subject of a current Denver Research Institute study.

Undesirable Substances That Remain After Conventional Treatment May Present Technological Barriers To Reuse. Some reuse applications result in the concentration of an undesirable substance or result in potentially more serious problems than occur in a series of successive uses alternating with discharges. The clearest examples occur in potable reuse applications, where conventional wastewater treatment is ineffective in removing heavy metals, organic compounds such as estrogen, and slow viruses such as hepatitis. Direct potable reuse following conventional treatment is unsafe because of the risk of concentration of metals and organics and the risk of virulence of slow viruses. Although all such contaminants exist in sewage effluent, conventional treatment followed by release into a river is believed acceptable because of the time delays and dilution that occur before intake into the next potable water system. Another example occurs in power plant cooling applications, where salt (TDS) concentrations occur following each reuse cycle. As treatment to remove TDS is uneconomic, the cooling water is replaced and disposed of after it reaches an undesirable level. The only potential solutions to these technological barriers also are technological: to develop new treatment processes that can effectively remove the contaminants at an acceptable cost. It should be noted that, except for potable reuse, the presence of undesirable substances does not necessarily inhibit reuse. The significance of the barrier must be evaluated on a case-by-case basis. Some reuses will be prohibited while in other cases the reuse is possible but the economic feasibility will be altered by the technological problem.

CONCLUSIONS ON STRATEGIES TO PROMOTE PLANNED REUSE

As the prior sections have discussed, there are numerous legal, institutional, political, economic and technological barriers to planned reuse, and not all of the barriers can or should be overcome. Some reuse barriers, however, run counter to the public interest and some of these might be overcome by one of the following strategies.

Use Of Imaginative Tactics By Prospective Reusers. Certain barriers to reuse have been overcome by some reusers who have applied imaginative tactics. In instances where the barrier itself serves a useful social purpose, as is true with the traditional water rights systems of the CRB states, it clearly makes more sense to achieve reuse by tactical means while leaving intact the policy or statute that indirectly presents a reuse obstacle. Where state water law restrictions on changes in type of use, point of diversion or stream regimen inhibit a planned reuse, the proposed reuser can negotiate for a water right that could accommodate the plan, or modify the plan to minimize the need for radical changes in the water right, or negotiate with other water users to compensate them for the damage caused by such changes, through purchase of compensatory water rights or by payment of indemnification. Similarly, when reuse plans are indirectly blocked by federal and state environmental legislation, the proponents should have thoroughly researched environmental impacts, studied possible alternatives for feasibility, and planned mitigating actions, as well as have emphasized the favorable environmental effects of the reuse.

Strategic Actions By The State Water Allocation Agency. State water allocators (e.g., State Engineers) can help promote planned reuse projects in several ways, involving use of their administrative flexibility (within the limits established by legal and judicial constraints) in making water allocation decisions, or involving use of their considerable influence in encouraging reuse plans. Use of these strategies by a water allocator is properly limited to cases where the public interest would be served by planned reuse, either to reduce the amount of consumptive use of scarce water supplies, or to improve water quality by consuming low quality water rather than better water. In such cases, a water allocator might:

- when an appropriation of new water is sought, and no return flow to the river is practicable, narrowly define the extent of the appropriative water right to the minimum amount that would be required when a planned reuse method is employed, rather than allocating a more generous quantity of water;

- when a water right transfer is being sought, and the new user seeks to consume a quantity greater than would be consumed under planned reuse, narrowly define the extent of the right to encourage reuse. (This could be done, in some cases, through administrative interpretation of the "beneficial use" and "prohibition of waste" statutes in western water law);

- when a new or changed water right is sought for a planned reuse that appears to promote the public interest, utilize special measures to provide expert advice to the applicant on actions that would simplify the process of obtaining necessary water rights, or would reduce the applicant's uncertainties or risks;

- when water rights disputes are likely to delay or interfere with a planned reuse that promotes the public interest, provide special services to help mediate such disputes or propose mitigating actions by the reuser that could avert injuries to other water users.

Strategies Involving Changes In Statutes That Bar Reuse. Certain federal and state statutes indirectly block plans for reuse. Although in most cases, the statutes provide offsetting advantages, there are cases in which new legislation, or amendments, would appear to promote reuse while preserving the essential advantages of the current statutes. One example is that of the pervading uncertainty over federal and Indian reserved water rights, that inhibits western water resources development. Possible solutions include new congressional legislation that would: (a) assume federal responsibility for litigation to adjudicate or quantify reserved rights on rivers where pending federal-state rights conflicts exist; (b) provide federal indemnification to state water rights holders in case of preemption; (c) provide federal "disaster" loans to state water rights holders in case of preemption; or (d) abandon all federal reserved water rights. A second example involves abuse of certain environmental protection statutes as a device to block development; this has resulted (intentionally or not) in blocking water resources projects that involve planned reuse. In such cases, a strategy of seeking congressional support for amendments to the statutes seems warranted, either to permit appeal to an interagency review committee (as was recently done with the Endangered and Threatened Species Preservation Act) or to permit exceptions to be made by a state governor when certain conditions have been met. A third example involves state statutes governing the process of obtaining approval for a change in water rights. Although no radical or fundamental changes in the water rights procedures seem warranted, continuing attention should be given to improving the procedures. Improvements are desirable that would reduce unnecessary delays, reduce the need to incur significant legal costs while pursuing changes in rights, and minimize the uncertainty as to the extent of the water right that will ultimately be defined as a result of the process. All of these imperfections can be barriers to planned reuse.

Strategies To Reduce Institutional Conflicts Blocking Reuse. Inherent conflicts between water allocation and water quality agencies (both state and federal) that inhibit planned reuse can be reduced through better interagency cooperation. Increased recognition of the value of planned reuse in solving both water quality problems and some water quantity problems will help bring about better interagency cooperation and identification of more areas of compromise that will promote reuse. Proponents of planned reuse can help weaken institutional barriers to reuse by promoting recognition of planned reuse as a matter of public interest, and thereby influence state water policy (as set by the governor and the legislature). The greatest opportunity for influence may come through intervention by advocates of planned reuse in conflicts among parties-at-interest over development plans involving water reuse. Constructive intervention will involve assuring that the benefits of planned reuse are made visible to the contending parties and to state officials who may have authority to resolve the conflict.

ACKNOWLEDGEMENT

This paper is derived from a Denver Research Institute, University of Denver study funded by the Office of Water Research and Technology, U.S. Department of the Interior, under Agreement No. 14-34-0001-7816. The full study, to be published in April 1979, is: J. Gordon Milliken, Loretta C. Lohman, Anthony S. Trumbly, and Lynne Roll, *Overcoming Legal and Institutional Barriers to Planned Reuse of Water in the Colorado River Basin*, Washington, D.C.: Department of the Interior, Office of Water Research and Technology, 1979.

REFERENCE

1. Department of Justice Appropriation Act for 1953 (Act of July 10, 1952), § 208, 43 U.S.C. 666 (1970) (popularly known as the McCarran Amendment).

PUBLIC PARTICIPATION IN THE ADOPTION OF WASTEWATER RECLAMATION PROJECTS

William H. Bruvold
University of California
Berkeley, California

James Crook
State of California
Department of Health Services

Introduction

The Federal Water Pollution Control Act of 1972 gave impetus to wastewater reclamation and reuse by requiring secondary treatment for most municipal wastewater (Walker and Cox, 1978). This requirement, when added to the need for additional supply in semi-arid regions of the nation experiencing continued population growth and increasing infeasibility of large interbasin transfers of water, has made wastewater reclamation an idea whose time has come. Coupled with Public Law 92-500 was the provision that the Federal Government would pay up to 75 percent of the construction cost for approved wastewater treatment facilities. In California the Clean Water Bond Issues of 1970 and 1974 can cover another 12½ percent of the cost of constructing wastewater treatment facilities and thus communities are required to provide only 12½ percent of construction costs from local funds (State Water Resources Control Board, 1976). Such developments bring reclamation options clearly to the fore. Serious consideration of reclamation and reuse versus treatment and discharge now seems imperative since secondary treatment is mandatory, since a good deal of the construction costs may be covered by federal and state funds, and since a little extra treatment beyond the conventional secondary treatment would make the reclaimed water usable in a way that would require a lesser supply of additional fresh water.

The Federal Clean Water Act of 1977, Public Law 95-217, amended Public Law 92-500 and has provided even more incentive for wastewater reclamation and beneficial reuse. The Clean Water Act will increase from 75 to 85 percent of construction costs for wastewater treatment facilities covered if new and innovative post-secondary treatments are included. With state contributions at 12½ percent this means that a local community might have to provide only 2½ percent of construction costs if more advanced forms of wastewater treatment are undertaken (EPA, 1978). Additional incentives for reclamation are provided by pretreatment requirements, Section 307 of Public Law 95-217. These requirements are designed to keep some 65 toxic and carcinogenic substances from discharge into publicly owned treatment works, thus making treated effluents freer from serious pollutants that wastewater treatment plants have difficulty removing by standard operations. Further, California requires that option consideration include reclamation and reuse (SWRCB, 1976). Finally, Public Law 95-217 still requires secondary treatment of wastewater for most municipalities, even though the time for completion of the treatment facilities may be extended into the mid-1980s. Thus the incentives for municipalities and communities to consider and adopt wastewater reclamation and reuse are even stronger now than in the early 1970s. But how do municipalities decide for or against reclamation and beneficial reuse?

How is the voting public involved in these decisions and is their involvement constructive or does it cause problems? This paper addresses these questions and turns next to a consideration of the community decision process on wastewater treatment.

Community Decisions on Reclamation

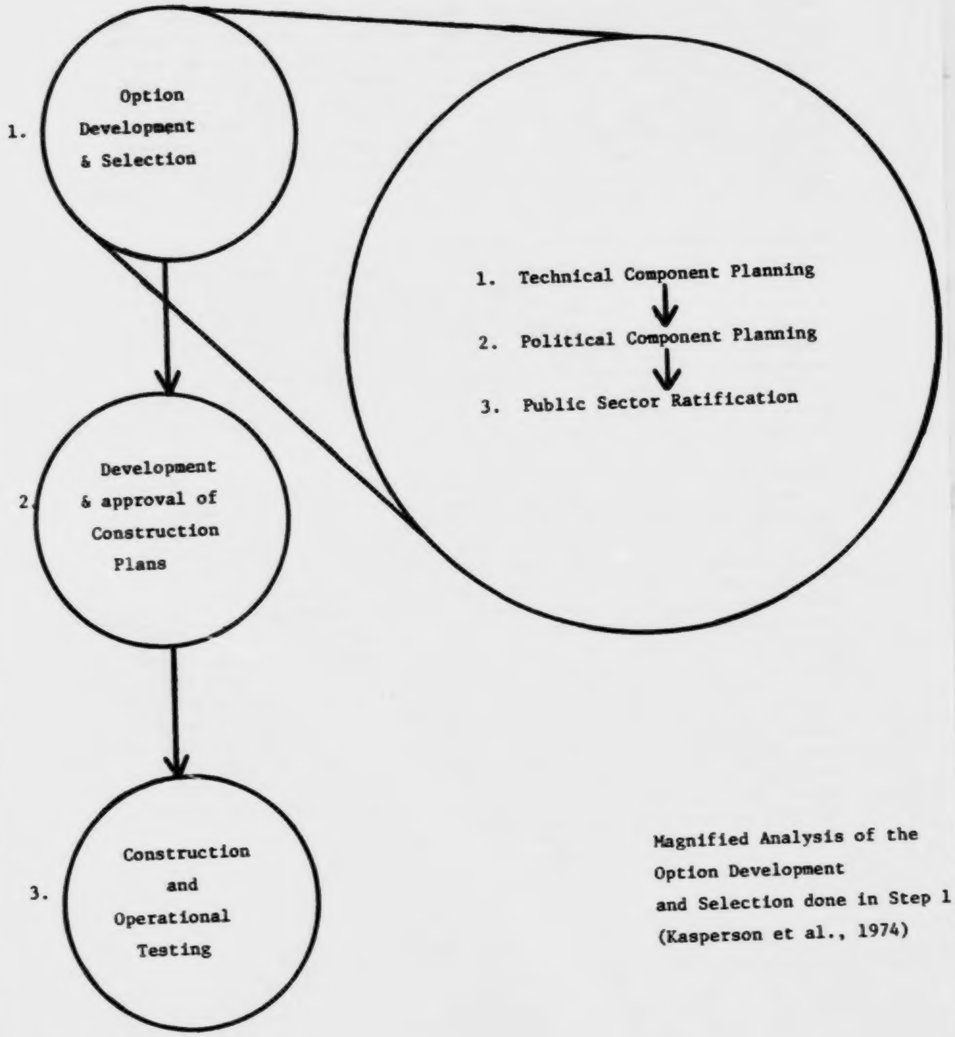
The State Water Resources Control Board (SWRCB) directs federal and state funds to California communities for the construction of wastewater treatment facilities. This agency has produced a large and detailed compendium of guidelines for communities applying for construction grants (SWCRB, 1976). There are three major steps in the process of obtaining construction funding outlined in Table 1: first, option development and selection; second, development and approval of construction plans; and third, actual construction and operational testing of the wastewater treatment facility. Our focus here is upon step one, option development and selection.

Kasperson, Baumann, Dworkin, McCauley, Reynolds and Sims (1974) have written a useful monograph on community adoption of water reuse systems. In essence, they report that step one noted above is itself usually divided into three sequential steps of considerable political and social complexity. In the first step the technical planning sector comprised of city employees and consulting engineers select one wastewater treatment option that in their opinion best meets the water resources situation, perceived public goals, internal agency goals, and the perceived goals and opinions of significant others in the community decision making structure.

The one option so selected is then passed over to the local political decision making actors who themselves test it against the water resources situation, perceived public goals, local internal political goals, perceived public opinion, and involved state agency opinion. If the option survives the local political analysis it is ready for presentation to the local voting public for endorsement through public hearings, city council votes, and most importantly, local bond issue approval to underwrite the local share of construction costs. Kasperson and others (1974) did not have a great deal to say about the public ratification third step because at the time their report was prepared very few if any major reclamation and reuse projects had been actually adopted in the United States. Thus questions arise concerning this third step. How can the public best be included and involved in the adoption process? How have initial efforts at adoption fared? Can anything be learned from early efforts that would extend the Kasperson and McCauley model on how adoption occurs and how this process might be facilitated?

Table 1

Schematic Outline Showing Community Adoption of Wastewater Treatment and Reuse Projects


1.
Option Development & Selection
2.
Development & approval of Construction Plans
3.
Construction and Operational Testing
1. Technical Component Planning
2. Political Component Planning
3. Public Sector Ratification

Magnified Analysis of the
Option Development
and Selection done in Step 1
(Kasperson et al., 1974)

Major Steps Formally
Required in California
(SWRCB, 1976)

Santee and Livermore, California

Kasperson and others (1974) make much of the well known early innovations of Santee, California in wastewater reclamation and reuse. They do not report, however, that a recall election was mounted against two directors of the Santee County Water District in 1969 and that before the recall election was held the internationally known manager of the district left to take another position. The issue of the recall election was increased sewer charges levied against businesses and commercial establishments in this unincorporated area of San Diego County. Today, the Santee area is connected to the San Diego regional sewer system and its once famous advanced wastewater treatment plant operates at less than one-quarter capacity just to service three small recreational lakes. The swimming pool once filled with reclaimed water is now filled with tap water. The tree farm which used reclaimed water has failed. The golf course that once used reclaimed water heavily now relies on other sources and reclaimed water serves only as a standby source. These developments in Santee seem not to be widely known but they do attest to the importance of having public support for the cost of advanced waste treatment and reuse options chosen by the technical and local political sectors. The main problem at Santee seemed to be the economic, rather than the health or the environmental impact.

Another relevant example is provided by Livermore, California. During the summer of 1976 we did some preliminary survey work in Livermore in order to pretest and refine the interview schedule that was subsequently used in the major survey work reported later in this paper. We interviewed 24 respondents in Livermore during July of 1976. This small sample in no way comprised a representative sample of the voting population of Livermore. They were selected from only six city blocks well distributed across the city in order to provide a good test of our interview procedures. However, at the time, a rather puzzling and disconcerting result developed. 71 percent (17) of our sample of respondents said they did not like, and were opposed to, giving their local wastewater secondary treatment and then exporting it to San Francisco Bay by a large pipe line that some called the "super sewer." General revenue bonds to support the construction of the pipe line and treatment plant were voted in on November of 1976. The Livermore vote on this issue was 6661 "yes" and 9791 "no." Thus our August 1976 poll, rudimentary as it was, predicted the 1976 November vote on the bond issue with surprising accuracy. Other treatment and reuse options were available for Livermore, one involving creation of a local reservoir coupled with agricultural irrigation, and another discharge into Alameda Creek used locally for waterbased recreation. Would Livermore not have experienced a failed bond issue for construction of their sewage treatment plant if they had selected one of these options instead of conventional treatment and export to the Bay? How might a clearer reading of public sentiment in this issue have been obtained? The third stage of the Kasperson and McCauley model is indeed important for community adoption and the remaining sections of the paper will consider this matter in some further detail.

The Optimal Time for Public Input

It could be argued that the best time for public input into the decision making on wastewater treatment and reuse would be early in the process, stage one of the Kasperson and McCauley model, in order to be sure that planning would proceed on a course acceptable to a majority. Democracy is based upon the assumption that governance is best when based upon the active involvement of an informed populace, and this principle implies that the people affected should decide early. Thus, it would be in principle, wrong to put off or delay the decision of the people unless we wish to thwart their will. On the other hand, it may be argued that the ordinary voter lacks the technical expertise to formulate options for wastewater treatment and reuse and then decide, on his own, which is best. Further, it may be argued that most people, even if fundamentally capable of understanding the necessary basics of technology involved, are not interested in, or do not have the time, to on their own develop and assess options and then decide what is best by some systematic means of analysis. Thus it seems that pure democratic principles and lack of technological expertise on the part of the voting population create a problem for early involvement of voters in the decision process. Democracy is made liable to critical attack if voters make uninformed decisions in ignorance of, and apathy toward, the issue at hand.

Following this line of thinking, it might be argued that public involvement should come later in the decision making process where the option selected by technological experts is ratified as in the vote on a local bond issue, stage three of the Kasperson and McCauley model. This procedure allows technological and professional experts to do the planning and analysis work that requires technological and professional expertise. These individuals select and develop options, decide which of several options is best, and then present the one selected to the voting public for ratification in a yes-no referendum. Here, only one option is presented and the voters have the choice of affirming or denying support to the one choice the technological experts and politicians themselves have chosen. Such an approach, it may be argued, preserves democratic principles and allows technological personnel to do technological work.

All of this sounds pretty good, but unfortunately it may not work very well. The experiences of Santee and Livermore, California, provide examples of how technical experts and political planners can misjudge the public will. Further, local bond issues of any type, wastewater treatment included, will likely be very hard to pass in this era of double digit inflation and Proposition Thirteen style taxpayer revolts. Thus the technical experts and professional planners must be extremely careful to choose wastewater treatment and reuse projects that have a great deal of public approval just to get the vote needed, whether a simple majority or a two-thirds majority, to finance construction. Also, it may be argued on more theoretical grounds that the bond issue vote on one option chosen by the experts weighs too heavily in favor of technocratic expertise and too lightly for democratic principles. Most voters likely are not expert enough to develop and assess options, but they may well be able to clearly understand the options and option analysis developed by the technical experts when this information is presented using non-jargonistic language. Further, they might even find such issues interesting since the options apply to their own community and their own local taxes.

Perhaps there is a third way to proceed in this area, a way that will more properly balance technical and democratic imperatives while also pragmatically insuring more success at the polling booth during local elections. Perhaps the technical and planning sector should select three or so options for wastewater treatment and reuse that are feasible for the area and which deliberately represent divergent or or different solutions for reclamation for that area. Then careful comparative analysis of the options could be developed. Once option development and analysis were completed by the technical experts and planning professionals, these data could be carefully translated into lay language and presented to voters for their reaction and input before going to a formal bond issue vote. Such a procedure would not require each voter to become expert in wastewater reclamation and reuse but it would also not put off voter involvement until the eleventh hour. However, is this idea practical? Can it be done, and if so, will it be successful? We have some recent data from a California study to which we now turn that bear upon these questions.

Preliminary Results from the Basin Planning Study

The Water Resources Center of the University of California and the Office of Water Research and Technology of the United States Department of Interior have sponsored a study done by Dr. Crook and myself that deals with public opinion on wastewater reuse options developed for various communities throughout California. The data gathering phase of this research was just very recently completed and it is neither possible nor proper to cover all results obtained in this report. However, some basic findings can be described and, more importantly, our study methods can be presented since these bear on the question of how to present several wastewater treatment and reuse options to a voting public.

Our California study involved ten cities for which clearly different wastewater treatment and reuse options already existed. Usually, but not always, three general options existed: (1) conventional secondary treatment and discharge to the environment with little if any beneficial use in the community; (2) secondary treatment plus additional treatment to make the reclaimed water usable in the local area for purposes such as recreational impoundments, landscape and greenbelt irrigation, or agricultural irrigation; and (3) secondary treatment plus extensive additional treatment such as coagulation, sedimentation and filtration or reverse osmosis in order to make the water suitable for park and schoolyard irrigation, high contact water sports, or recharge into an aquifier used downstream for domestic supply.

One hundred and forty respondents of voting age were selected from each city studied by scientific sampling procedures designed to produce a fair and unbiased sample of voter opinion on the three options. And here is the interesting part: a summary of each option was presented to each respondent that described in general terms the kind of treatment given and the use to which the reclaimed water would be put. Then the environmental impact of each was presented, then the health impact, and finally the economic impact was presented for each option. This rather lengthy presentation of options and their predicted impacts represents a departure from usual survey research procedures in that it requires the interviewer to conduct an educational presentation in the middle of each interview. Nevertheless, the procedure worked and allowed us to then ask respondents which option they liked best, which they liked least, and whether they would vote yes or no on each plan if it were presented in a local bond issue referendum.

Undoubtedly you all have many questions about the details of this unusual type of survey research. We plan to spell out all these details in subsequent publications. Let us just say here that each set of option statements was carefully scrutinized by a panel of knowledgable individuals before the survey research was conducted to insure that each was complete, unbiased, fair and factually correct. Also, respondents were made to discuss the three plans in some detail before answering the major questions by saying which environmental impact they liked best and least, which health impact they liked best and least, and which economic impact they liked best and least. Results by community for generalized option types, realizing that all options don't exactly fit the classification scheme, are presented below for plans liked best and least, and for current voting predictions on each plan for the first three communities studied.

Non-Binding Referenda

The survey research approach of the California Basin Planning Study represents a viable procedure for obtaining public reaction to several options developed by the technical planning sector. A random probability sample of voters does not involve the entire affected voting population but the results are representative of the entire voting population. The brief educational effort provided by the interviewers is not extensive but care can be, and was, taken to make it fair, factual, complete and systematic. However, would there not be a way to involve all voters in the intermediate decision making process after all had been exposed to an informational campaign and debate of the several options developed by the technical and planning sectors? We think that it would be a most valuable experiment in public participation to present information on several wastewater treatment and reuse options followed by a non-binding referendum conducted in connection with a local or county election. The idea here would be to mail to each voter information that carefully and systematically described the treatment and use for three or four viable options. Additionally, comments on environmental, health, and economic impacts associated with each option would be described. All of this would be accomplished using lay language with simple maps and charts as needed. Position statements on various options by opposing groups could also be included as part of the packet. Certainly in California, with its many state and local voter initiatives, such as the now famous Proposition Thirteen, there is precedent for this kind of mailing to each registered voter.

The mailing, as we envision it, would be accomplished and followed by public meetings, media coverage of debates, newspaper articles, and the like, to arouse interest in the matter and to supplement the written information provided each registered voter. At the polls, say at the end of the regular election ballot, would be the non-binding referendum where each voter would be asked to indicate yes or no for each option. Such results could then be considered by the technical and planning sectors for choosing one option for a formal bond issue vote. As far as we know, such a procedure has not been tried before. It is new, it would cost money, and it would reduce the decision making prerogatives of the technical and planning sectors. On the other hand, it might be much less costly in money and confidence in public officials than failed bond elections such as the Livermore, California example cited earlier. Further, such an approach could markedly increase chances of local election bond issue approval, it could increase confidence in public officials and their technical planning consultants, it could increase citizen participation in local political decisions, and it could serve as a model for citizen participation in school bond issues, planning and zoning decisions, public works improvements and other similar matters requiring voter ratification at a local or county election. Thus we see a two-step process in the third or public ratification stage recognized by Kasperson and McCauley. The first step would involve assessing public opinion regarding several viable treatment and reuse options. The second would involve formal voter ratification of the one plan selected as best for the community by a decision process that has appropriately involved the technical and planning sectors, local political leaders and the voting public.

Table 2

Wastewater Treatment and Reuse Options Liked Best and Least by Respondents from Three Cities Surveyed *

City Surveyed		Generalized Option One **	Generalized Option Two	Generalized Option Three	No Answer
1. Livermore	Best	14%	72%	12%	2%
	Least	64%	4%	28%	4%
2. Watsonville	Best	7%	80%	12%	1%
	Least	62%	4%	32%	2%
3. San Luis Obispo	Best	5%	56%	35%	4%
	Least	86%	2%	8%	4%

* There were 140 respondents for each city surveyed.

** Generalized options are described in the text.

Table 3

Yes and No Voting Preferences by Wastewater Treatment and Reuse Option Category *

City Surveyed		Generalized Option One **	Generalized Option Two	Generalized Option Three
1. Livermore	Yes	21%	71%	31%
	Unsure	2%	5%	3%
	No	77%	24%	66%
2. Watsonville	Yes	30%	84%	30%
	Unsure	3%	3%	4%
	No	67%	13%	66%
3. San Luis Obispo	Yes	14%	76%	62%
	Unsure	5%	6%	6%
	No	81%	18%	32%

* There were 140 respondents for each city surveyed.

** Generalized options are described in the text.

References

1. California State Water Resources Control Board. Clean Water Grant Program Procedures. Sacramento, California: State of California, 1976.

2. Kasperson, R. E., Baumann, D., Dworkin, D., McCauley, D., Reynolds, J. and Sims, J. Community Adoption of Water Reuse Systems in the United States. Washington, D.C.: Office of Water Resources Research, United States Department of the Interior, 1974.

3. United States Environmental Protection Agency. Pretreatment of Industrial Wastes: Joint Municipal and Industrial Seminar. Cincinnati, Ohio: United States Environmental Protection Agency, 1978.

4. Walker, W. R. and Cox, W. E. Law Literature Review for 1977. Journal Water Pollution Control Federation, 1978, 50, 1689-1694.

* This research was supported by Grant Number W-515 from the University of California Water Resources Center and by Grant Number 14-34-0001-7813 from the Office of Water Research and Technology, United States Department of the Interior.

A WATER UTILITY'S CONSIDERATIONS FOR IMPLEMENTING A WASTEWATER REUSE DUAL WATER SYSTEM

William G. Richards
Assistant Chief, Research and Development
Philadelphia Water Department

Patrick R. Cairo
Chief, Research and Development
Philadelphia Water Department

Arun K. Deb
Principal Environmental Systems Engineer
Weston Environmental Consultants

BACKGROUND

Dual water distribution systems have been proposed as an alternative to conventional water systems primarily with the objective of conserving limited supplies of good quality water (1). This is accomplished by supplying all or part of the non-potable supply needs with water of less than potable quality. It is also conceivable that dual distribution systems could become economically attractive when the costs of additional water treatment required to make a poor quality water source potable become comparable to the costs of a second distribution system. In either situation the non-potable water supply may come from an entirely different source, from the same source but with a lesser degree of treatment, or may be reused water.

As a result of environmental concerns and subsequent federal regulations dramatic changes have taken place over the last decade in the degree of treatment provided for wastewater effluents. The present research and investigation into potential long term health effects that may occur due to trace organic compounds identified in water supplies and finished water promises a similar evolution in the water treatment technology. A major impact facing public and privately owned utilities is that treatment related costs for both water and wastewater will represent a substantially larger fraction of their total budget than ever before. Under these circumstances some long term practices in sanitary engineering may deserve economic re-evaluation. Dual distribution systems and water reuse may be found to be viable economic alternatives under certain conditions.

At the present time the overwhelming practice of water utilities is to provide the entire supply at a quality sufficient for the highest intended use, namely potable water. In general, the sole exception to this condition would be when an industry requires additional treatment to prevent corrosion or scale formation in some of its processes. However, this is handled almost without exception by the industry doing additional treatment after receiving a single water supply from the utility. The reliance upon a single high quality water supply is primarily a result of the relative disparity that has existed between the costs of distributing water as compared to the costs of securing and treating it. Distribution costs have simply been too dominant to allow consideration of multiple quality supplies even though less than 5% of the volume being delivered may require potable level quality. However, under the presently developing conditions of increased competition for water supplies or substantially increased treatment requirements, there may develop a change from the previous dominance of distribution costs vis-a-vis treatment costs. Under these circumstances it may be fould that for some areas supplying more than one quality of water would be a viable economic alternative.

In a dual distribution system, potable water would be used for supplying those needs where water of high quality is desired, and might also provide some portion of the remaining water demand. It would not be necessary or even advantageous to preclude the use of potable water from all non-potable uses. Rather, the actual distribution between the two different water systems would depend upon factors, which include economics, acceptability, site specific conditions determining where the demand for each type of water is centered, and the quality of the non-potable supply. The variety of conceivable dual supply systems is numerous. Alternatives could range from limiting the non-potable supply solely to industrial, irrigational, or fire supply purposes to using the potable supply only for drinking and cooking purposes. Within this range the balance between potable and non-potable supply will depend upon a community's residential, industrial, commercial, and public demands and thus the possible combinations are numerous.

In a related field, the advancement of wastewater treatment technology and federal requirements have brought about the improvement of wastewater effluents to a point where they can now be considered a potential resource. If this fact is too radical to grasp, a complimentary argument can be made that the costs involved in cleaning up wastewater have made effluents too expensive to simply cast away. The fact of the matter is that in many areas the quality of effluents from advanced wastewater treatment plants is comparable to the quality of good surface water supplies. Certainly it is practicable that some wastewater effluents are at a level where they are useable or can readily be made useable for lesser quality water needs.

Thus the potential for combining the concepts of reuse and dual water systems becomes evident. The reuse of wastewater treatment plant effluent, receiving additional treatment as required for its intended need, is one type of dual distribution system which deserves serious consideration. Reuse could, for example, provide the non-residential portion of a community's water supply. In water-short areas this combination could preclude the need for total reuse of water for potable purposes, and would avoid controversies arising from the delivery of non-potable water to residential customers. In any event reuse provides excellent opportunities when the economic conditions are favorable for conserving limited water supplies.

Even when the concept of combining reuse and dual systems has become acceptable to a community there will still remain concerns of local officials that will have to be answered. As with all innovative practices, the proponents of a reuse dual system will have to take the initiative to show the benefits that will be incurred. It can be expected that the institutional systems established in the community and government will be reluctant to move to new ideas and will require considerable documentation of advantages as well as convincing that all disadvantages and potential hazards have been taken into account. This paper will later address the considerations that will have to be taken into account by a water utility contemplating such a system. It will also consider the problems that would have to be overcome before a reuse dual system could be implemented.

MULTIPLE SUPPLY SYSTEM STUDY

In order to investigate the feasibility for dual water systems a research study was proposed by the National Science Foundation (2). A grant (Multiple Water Supply Approach for Urban Water Management: ENV 76-18499) was given to Weston Environmental Consultants to conduct an economic analysis of various alternatives for long-term water supply management including multiple water systems. As part of this study basic water supply and demand data was collected and analyzed, water quality and treatment requirements were developed and optimizations of several design systems were produced. An efficient and practical systems model was developed to help planners determine if multiple supply systems will be beneficial or economic in long-term planning. This model is very flexible and can handle a conventional system, a dual or multiple system including reuse, or a regional system including several communities. It is a powerful tool for technical and economic analysis of various alternatives for long-term water supply management.

In multiple supply systems two or more grades of water would be supplied to consumers through separate distribution systems, according to the quality and quantity requirements for various uses. For the purposes of that study three grades of water were defined. "Potable water" was defined as being completely safe for long-term continuous human ingestion that would satisfy primary and secondary regulations of the Safe Drinking Water Act including projected regulations on natural and synthetic organic compounds. "Subpotable water" was defined as water that is bacteriologically safe, but may contain trace metals and organics, and would not be considered safe for long-term continuous human ingestion. A third category, "Non-potable water", was defined as water not intended and not safe for ingestion. The definitions will be used through the remainder of this discussion. Under these definitions potable water would be used for the limited occasions when water of high quality is required. Subpotable water, when used, could be available as a supply for all household uses other than potable. Non-potable water would primarily be used for industrial and irrigational uses.

This study showed that it is economically feasible to consider dual water supply and wastewater reuse as viable alternatives for conventional water management under certain circumstances. A variety of hypothetical conventional and corresponding dual water supply systems, including reuse, were examined. Preliminary design was performed to determine a cost of collection, transmission, treatment, and distribution and to compare alternatives on a present worth economic basis. It has been found that, in general, if activated carbon is required to produce potable water then the cost of a dual water system will be comparable to the cost of a conventional system. In the case of limited reuse for a community's non-potable uses, the cost of a dual system may be cheaper. If effluent from a secondary treatment plant followed by filtration and chlorination is available for reuse and required for quality reasons, the cost of a dual system may be comparable to a conventional system cost.

In addition to the many hypothetical cases, the study considered two actual test case evaluations which considered reclaimed wastewater as part of the dual water system alternatives for non-potable uses. The two case studies were Tampa, Florida and the Eastwick Urban Renewal area of Philadelphia, Pennsylvania. Tampa, Florida has a present population of 450,000 which will increase to 600,000 by the year 2000. The Eastwick area of Philadelphia, Pennsylvania is undergoing urban renewal which will increase the population from about 20,000 to 37,000 by 1985. Both areas have existing water systems which will have to expand to handle the growth and demand for potable water. For Tampa a wastewater reuse dual water system was comparable with the conventional alternative while a non-reuse dual system exceeded the cost of the conventional alternative by about 50%. For the Eastwick test case, a non-reuse dual alternative was found to be comparable to the conventional alternative while a reuse dual system requiring filtration and chlorination was about 60% more expensive. It was noted in the Eastwick case that site-specific conditions prevailed and that a reuse dual system might be more attractive in the vicinity of two other Philadelphia wastewater treatment plants where a greater industrial and irrigation demand for the non-potable supply was believed to exist. It is this potential that will be receiving consideration by the Philadelphia Water Department in the near future, and is the proposed scheme for this discussion. More information on this study will be given by Arun K. Deb in his paper, "Water Reuse Management Using Dual Water Supply", to be presented later in this Conference.

REUSE BENEFITS

Wastewater reuse presents several advantages to a water utility, whether located in areas of water shortages or in relatively water-rich areas of the United States. The good quality water, which would have otherwise been used for the water consumption served by reuse, would be conserved for future potable use. If the good quality water is transported over extensive distances before it is used, reuse would reduce the cost of transportating an equivalent volume of water from that distant source. A similar situation would exist when water is withdrawn from a less than excellent quality source and extensive treatment is required to achieve a potable supply. Reuse would reduce the costs of securing and treating this equivalent volume of water.

The reuse of wastewater would also reduce the total volume of effluent eventually discharged to water courses by the amount of reuse. This reduction in effluent would have an incremental benefit in terms of improved water quality to the receiving stream in that it would be better able to assimilate the lowered discharge volume. In addition the municipality would benefit in that it would find it easier to meet regulatory effluent requirements based upon a total pollutant load allocation system. It is conceivable that additional advanced treatment due to increasingly stringent effluent regulations could be avoided by adoption of reuse.

Assuming that a sufficient demand for reused wastewater exists, the income from the non-potable supply would generate additional revenue. In the comparisons between conventional and reuse dual systems previously discussed no distinctions were made between water and wastewater utilities. Only the overall economic advantages that might occur were considered. In reality most of the savings would be incurred by the water utility while the wastewater authority would take on additional expenses. It is assumed that an equitable arrangement between the two utilities could be established to rearrange rates for the two grades of water and for wastewater treatment. It is conceivable that once a reuse system was instituted and available at a cheaper price than the potable grade of water, a demand above that anticipated could develop allowing for additional revenue to be generated. In Colorado Springs, reused wastewater is distributed and sold at a price two-thirds that of the potable system.

It should certainly be appreciated that reuse also takes advantage of a potential resource that would otherwise be discarded. The value of this resource would be most appreciated during a period of extreme drought when other sources of water would become severely limited. In addition, indirect reuse is already a fact of life throughout the world. During low flow conditions the total withdrawals from the Schuylkill River are five to seven times the total flow by the time it reaches Philadelphia, where it is used as the source of potable water for about half of the City.

ECONOMIES OF WATER TREATMENT

Any decision concerning the implementation of a wastewater reuse dual system is dependent upon many factors including economics. In Philadelphia a prime consideration will be the eventual cost of the water treatment methodology that will be required for the removal of trace organic contaminants from drinking water. The City secures its water supplies from two surface waters both of which are subject to discharge of a wide variety of chemical and municipal wastes. Both sources are also frequently the site of serious industrial chemical and oil spills. A large number of organic chemical compounds have been identified by the City and by regulatory agencies in the river and the presence of some of these contaminants have been detected in trace quantities in the finished drinking water. When the Environment al Protection Agency (EPA) finally promulgates Primary Drinking Water Regulations for organic chemicals, either in the form of maximum contaminant levels or by prescribing specific treatment techniques, it is expected that Philadelphia will have to incur substantial additional treatment costs in order to comply.

Because of the uncertainty of the final regulations which are to be promulgated by EPA, the exact scheme of Granular Activated Carbon (GAC) or other treatment methodology is still unknown. In order to respond to these uncertainties and still be able to institute a treatment method within a reasonable period after regulations are in effect, the Philadelphia Water Department has constructed an advanced water treatment pilot plant. This pilot plant is capable of examining feasible treatment methods to allow an evaluation of their effectiveness and to generate design parameters for full-scale treatment design. Treatment methods under investigation have included use of GAC in modified filter beds, in downflow contactors and in sequence with ozone in the so-called Biologically Activated Carbon (BAC) process. Changes in the present treatment scheme, such as the use of disinfectants other than chlorine, or the application of disinfection at different points in the treatment scheme so as to avoid the creation of halogenated organics from precursors, is also being investigated. One fact that has become obvious as a result of this research is that the design parameters are going to be directly dependent upon the regulatory requirements that have to be met. Therefore until the EPA regulations are promulgated the final method of treatment will not be known.

The nature of the final organic regulations will be especially important to future water resource planning since there are considerable cost differences between the alternatives under consideration. The final costs will be dependent upon many factors, not the least of which will be what type of activated carbon will be used and whether it will be appropriate to be used in modified filter beds or in a series of contactors. Other factors that will be of considerable importance to the overall cost are the requirements for contact time and the regeneration frequency of the carbon. A major unknown at this time is the actual loss of carbon that can be expected to occur during regeneration under actual operating conditions. Another very substantial factor is the expected rate of inflation which will impact the O & M costs of GAC make-up and regeneration fuel costs. It is anticipated that the variability in costs between different alternatives for trace organic removal will be substantial enough to impact any economic analysis evaluating the attractiveness of a wastewater reuse dual system and a final evaluation will have to await determination of the specific water treatment plant improvements.

CROSS CONNECTION CONTROL

The establishment of a dual distribution system raises a concern regarding the possibility of cross connections between the two supplies. Although all states and many municipalities have established regulations concerning cross connections and in many instances may have an active program underway, these are rarely of a scale to be more than superficially effective. Thus the potential that exists for a cross connection to occur and the difficulty of establishing a means for preventing it must not be underestimated. The City of Philadelphia is presently attempting to institute a strong cross connection program. The program objectives are prevention of contamination of the water supply by containment at the point of supply with concentration on public facilities and private customers of potentially severe and moderate hazard. Advice on internal backflow prevention is available on request for private facilities but a total containment program for all private customers was too enormous for consideration. This program, as envisioned, would be capable with some modifications of regulating, inspecting, and overseeing a non-potable reuse system for non-residential purposes. The philosophy of this cross connection program would be inconsistent with, and unresponsive to, a dual system existing within residences.

In order to allow a dual distribution system, especially one using a non-potable reuse supply, a rigid local cross connection program is an absolute necessity. This program would have to be established and well underway before a dual system is initiated so that its administrators could participate and advise in the design of the system. In order to be consistent with most guidelines that exist within cross connection programs presently in effect, a non-potable reuse supply would be considered a severe potential hazard. This would require that any facility making use of both the potable and non-potable supplies would have to provide an air gap in its potable line so as to assure that the potable distribution system could not be contaminated in the event of cross connection or backsiphonage. In addition any facility using the non-potable supply should undergo an inspection for actual and potential cross connections and should install appropriate backflow prevention devices as recommended for the protection of its own personnel or visitors. Color coding of all fluid pipes should be required and internal regulations should be established and strictly enforced concerning connections from potable and non-potable lines. The cross connection administrators should develop an education program directed to users of a reuse system to alert them of the potential and actual hazards of cross connections.

The question of whether the local cross connection program administrators would be in favor of a municipal or utility operated reuse distribution system, even one excluding residential users, has not been answered yet. It should be anticipated that the initial reaction of such officials might be negative. However, in many areas there may be the existence of a comparable system that could show a favorable precedent. In Philadelphia, for example, there already exists within the Center City area a non-potable high pressure fire system which is served by pumping untreated river water whenever a multiple-alarm fire in the downtown area occurs. The establishment of a very close working relationship between the cross connection and wastewater reuse programs would be essential, since any serious incident soon after implementation would be disastrous to both.

PUBLIC RESPONSE

Other major factors to consider would be the response of public, industrial, and governmental sectors of society as well as increased regulatory requirements once reuse has been implemented. Numerous surveys (3,4) have been reported in the literature indicating that many applications of reuse are already acceptable to a large majority of the general public. In general reuse seems to be more acceptable in the western United States than in the eastern regions reflecting the latter's relative accessibility to alternate water supplies. The degree of public contact is also a major concern in the individual's perception of reuse. The further a reuse alternative is from drinking or contact uses the greater the portion of the public that is favorable to it. Sims and Baumann (3) report that 94-96% of the population respond favorably to reuse applications for supplying industry and for golf course irrigation.

Assurances that wastewater reuse would remain under strict control and that all possible steps are being taken to eliminate contamination of other supplies would probably be required to alleviate any fears that might develop within a community. This again points out the necessity for working with cross connection and public health officials to assure compliance by all those involved. To a small extent the degree of concern may be lessened if the quality of reuse required is high. Direct reuse of wastewater effluent without additional treatment would certainly require more concern than would a reuse application requiring filtration and disinfection.

Of course a primary factor in determining the public response to an industrial and irrigational reuse system would be the eventual impact on water and sewer rates. Recent rate increases in Philadelphia due primarily to a program to upgrade wastewater treatment in response to federal requirements brought on considerable opposition from the press. Even under the assumption that a reuse system would be implemented as a cheaper alternative to instituting a new treatment method for the entire water supply or for seeking out new, more distant source supplies, it would still be likely that increases over past rates would result. It could be a difficult task after the fact to explain to an aroused public that the increases to which they are being subjected are less than what would have occurred if a different alternative had been chosen. Thus it would be necessary to have an informed public aware of the decision process that lead to choosing a reuse option.

All of these concerns point to the necessity to create a greater awareness in the general public of the problems being faced by the water utility. If the public is to be expected to be responsive and approve of innovations such as reuse and dual distribution systems that will change the way they perceive their water supply, then they will have to be informed and perhaps even participate in the selection of choices that are available. If, on the other hand, the water utility managers perceive the public as an obstacle to water planning, and wish to limit or avoid contact with them, then they will find their options limited to those that will involve slight modifications or additions to the present operational systems and may eliminate potentially cost-effective methods. There is no question that a wastewater reuse dual system, even one directed primarily to industry, would require a great deal of public awareness and education. A substantial public information program would be required, and the cost of this program would probably be borne by the water utility.

INDUSTRIAL RESPONSE

It is a basic premise in consideration of an industrial reuse alternative that an industry would use the available water supply of satisfactory quality that has the lowest cost. Under this circumstance the location of a large industrial complex not requiring a high purity source of water in the vicinity of a wastewater treatment plant would seem ideal. However, the initial experiences of the Costra County, California, Reclaimed Water Project (5) show that this assumption may not automatically hold. Severe opposition to the introduction of reclaimed water as an industrial water supply was encountered, with no real reasons for objections being given by industry officials other than generalities about high operating and replacement costs from scaling and corrosion and apprehension as to the attitude of employees working near reused water. This initial reaction was overcome by the creation of a pilot plant which produced reclaimed water and the limited distribution of its product to demonstrate that the quality was sufficient not to create the anticipated problems. A potential area for demonstrating this fact short of a pilot distribution system might be to reuse effluent within the wastewater treatment plant for process purposes.

A major concern that will be part of industry's decision on whether to accept or purchase reused wastewater will be the degree of treatment provided by the utility and the resulting quality of the product available. Fynsk (6) has stated that "industry cannot be burdened with the cost of treating wastewater generated by others". The greater the degree of treatment provided by the municipality the less treatment, if any, that will have to be provided by industry. In deciding upon which water supply to use, the potable or non-potable source, industry will choose that whose total cost to them is least. If additional treatment required to bring the non-potable source up to a quality required by an industry is greater than the difference between the potable and non-potable sources, the industry is not going to use the non-potable water. The quality of the water required by industry will thus dictate the necessary degree of treatment and this will be a major determination of the cost of the reuse system.

In addition industry's own method of waste disposal and effluent requirements may impact their decision on whether to take advantage of reuse. If an industry presently has a discharge requirement of total dissolved solids which it finds difficult to meet the change to a reuse supply could substantially aggravate this problem. Similarly if the plant in question discharges to the utility's sewer system, it would have to seriously investigate whether the combination of reuse and its own practices would make it difficult to meet pretreatment requirements. On the other hand, a plant which presently discharges a relatively clean waste may find it advantageous to arrange with the utility to direct its effluent, without passing through the wastewater plant, to the wastewater plant effluent before final reuse treatment and distribution, and thus eliminating its own disposal requirements.

Another factor to be considered by both the industry and utility is the matter of reliability of both the demand and of the source. The utility must keep in mind that as market conditions change an industry's processes are constantly undergoing re-evaluation and change. As a result their water requirements may change drastically and rapidly. It would certainly not be wise to structure a reuse program around one large user whose use, by changes in processes or through failure, could vanish. Another consideration would be the availability of a continuous and near-uniform demand throughout the year as extra reused water storage requirements would introduce an additional burden. The industry, on the other hand, will want to be assured that both the quantity and quality of the reuse supply is at least as certain as the present potable system. A plant which depends on a minimum quality that is being provided by the non-potable supply is not going to react favorably to temporarily accepting either a drastically reduced quality of water or no water at all, should a failure occur in the reuse treatment system. (7)

A final consideration that will impact industrial response to such a program would be the number and degree of restrictions placed upon it by regulatory and municipal agencies. The potential requirements of a cross connection program have already been discussed. It should be pointed out, however, that if an industry would have fallen into a severe cross connection hazard category without reuse it would have had to comply with those requirements anyway. As will be discussed below, it can be expected that regulatory agencies will impose requirements in terms of liability, monitoring, quality control, and health assurance. How compliance with these requirements is divided between the utility and a cooperating industry may determine the acceptability of the reuse dual system.

GOVERNMENT AND REGULATORY RESPONSE

There are presently very few regulations from governmental and regulatory agencies concerning reclaimed water, basically reflecting the still limited status of reuse. It could be expected that as reuse becomes more prominent regulations would become both more prolific and more stringent. That certainly has been the history of regulatory agencies in other fields. These anticipated regulations would almost certainly increase the utility's or municipality's health-related responsibilities above those now usually required. It would be expected that extensive monitoring and inspection procedures would be a minimum requirement. Research or pilot studies might be necessary to demonstrate practicality and effectiveness. It might also be possible that until a good health and safety record were established, additional funds might have to be set aside as an escrow account against potential liability and health damage suits. (7)

It is perhaps an unfortunate commentary that the degree and intensity of the regulatory reaction to the concept of an implemented reuse system will probably be determined primarily by the degree and intensity of the public and industrial responses. A public outcry by citizens or environmental groups could cause regulatory agencies to be excessively restrictive in drawing up their regulations in order to avoid law suits directed at them. On the other extreme, a very favorable response by industry and resulting pressure on local governments to institute such a program with little or no restriction could lead to too lenient a safety program without sufficient attention to very real health concerns. It is also possible that wastewater regulatory officials would view reuse schemes as a means for dischargers to circumvent disposal requirements and lead to a corresponding reduction in allocation permits, thus resulting in the elimination of one of the perceived benefits of reuse to a utility. Should such a decision prevent a community from adopting reuse the anticipated improvement in stream quality would then be lost. Hopefully these types of reactions, though prevalent in the past, could be avoided and regulations based on merit and actual need would be established. However, it must be considered that the regulatory response would be the most uncertain and potentially could be the most important non-economic consideration to be taken into account.

PROPOSED STUDY

In the previously discussed dual water study, the wastewater reuse alternative investigated for the Eastwick area envisioned supplying secondary effluent from the upgraded Southwest Philadelphia Water Pollution Control Plant after it had undergone filtration and disinfection. This effluent would be pumped to a non-potable water distribution system for irrigation, industrial, and bulk users. The remainder of the water needs for this area would come from water underoing conventional and GAC treatment for the removal of trace organic contaminants. The Eastwick area was originally chosen to optimize a non-reuse dual water alternative and there is no large industrial water demand present in this area. Thus the reuse option examined did not turn out to be cost-effective in this study, but it was quite apparent that the results had been very site-specific, and that the reuse option would be found to be more attractive at other locations.

Both the Northeast and Southeast plants, Philadelphia's two other water pollution control plants, are located in close proximity to large industrial areas where the potential demand for reclaimed wastewater should be much greater than in Eastwick. Southeast is in close proximity to several petroleum refineries and a large municipal park and public golf course. Northeast is in the vicinity of a large industrial area of varied industries including several chemical plants. Both plants are undergoing extensive upgrading programs and will be producing an advanced wastewater plant effluent by 1983. Although investigations into the specific demand and quality requirements in these areas will have to be conducted, preliminary evidence and reaction indicates that the potential for an economically attractive reuse dual distribution system is much greater at either of these plants than it had been for the Eastwick study. This potential was considered sufficient by the Philadelphia Water Department to warrant a comprehensive study of the feasibility and economic potential for wastewater reuse at these two plants. If such a study should show promise it is hoped that by the time the upgraded plants are on line, most of the uncertainties relating to the considerations of trace organic removal methodology, cross connection program implementation and public, industrial and governmental-regulatory reaction will be better defined so as to allow a realistic determination of the potential for implementing a reuse dual distribution system.

REFERENCES

1. OKUN, DANIEL A., Planning for Water Reuse, Journal AWWA, 65:617, October, 1973

2. DEB, ARUN K., Multiple Water Supply Approach for Urban Water Management, Final Report on Research supported by the National Science Foundation under Grant Number: ENV76-18499, November, 1978

3. SIMS, JOHN H. & DUANE D. BAUMANN, Renovated Waste Water: The Question of Public Acceptance, Water Resources Research, 10:659, August, 1974

4. BRUVOLD, W.H. & H.J. ONGERTH, Public Use and Evaluation of Reclaimed Water, Journal AWWA, 66:294, May, 1974

5. FLETT, D.B., Wastewater Reclamation for Industrial Use, Journal AWWA, 67:75, February, 1975

6. FYNSK, A.W., Reclaimed Water for Industrial Use Excluding Food and Beverage Processing, Journal AWWA, 67:81, February, 1975

7. HEATON, RICHARD D., Economics of Reuse, AWWA National Conference, Atlantic City, N.J., June 27, 1978

BIBLIOGRAPHY

SHUVAL, H.I., Water Renovation and Reuse, Academic Press, N.Y., 1977

D'ITRI, F.M., Wastewater Renovation and Reuse, Marcel Dekker, Inc., N.Y., 1977

EAST BAY DISCHARGERS AUTHORITY
REUSE SURVEY

Daniel F. Murphy
Environmental Engineer
East Bay Dischargers Authority
22300 Foothill Blvd., Suite 602
Hayward, California

Garry E. Lee
Assistant Project Director
Lowry and Associates
25954 Eden Landing Rd., Suite 109
Hayward, California

East Bay Dischargers Authority (EBDA) has recently completed what is probably the most detailed (or finest-grained) survey for reclaimed water uses ever made in a highly developed metropolitan area. The specific objectives of our survey were to:

...Identify the location of potential reuses within EBDA's service area.
...Determine the economic and environmental feasibility of these reuses.

The survey was undertaken in the context of the 1972 and 1977 Federal Water Quality Acts, both of which encourage elimination of discharge of pollutants and reclamation of wastewaters. Motivation for the study was provided by the need to respond to both the general public and state and local elected officials on whether reclaimed water could be productively used. The general public asked this question at public hearings held to review EBDA's proposed major sewerage project. Elected officials asked the question under a variety of circumstances. These circumstances included direct questions by members of EBDA's governing Commission, conditions in permits required for the major sewerage project and by the Governor who established a State Office of Water Recycling. Motivation also came in the form of a two-year drought which underscored the fragility of California's water supplies.

In developing the scope of the reuse survey there was the option of performing a minimal survey, i.e., looking at a few large water uses, or of performing a fine-grained survey. EBDA elected to conduct the latter even though it might be initially more costly. If a minimal survey had been performed EBDA would have had to continually respond to questions of why didn't you look at this or that useage. Subsequent surveys would undoubtedly have had to be undertaken. EBDA, therefore, elected for the efficiency of performing the single technically and administratively integrated survey. This approach enabled dealing with all issues and interests in a short time frame. It provided a high level of credibility for the survey and undoubtedly saved EBDA from a constant fiscal drain for an endless series of minimal surveys.

From the outset of the survey EBDA was faced with two principle challenges: First, how to extract from the thousands of theoretically potential uses those relative few which would be viable. Second, how could the reuse program be integrated with the operations of other jurisdictions to EBDA's service area. It is the purpose of this paper to tell how these challenges were met and to offer suggestions for making similar surveys more effective.

GEOGRAPHICAL AND INSTITUTIONAL SETTING

Description of the physical and institutional setting in which the survey was conducted will assist in understanding the constraints within which it operated. EBDA is a joint powers agency formed by five local wastewater management agencies, two cities and three sanitary districts whose combined service area covers 200 square miles in southern Alameda County, California. This service area (shown in Figure 1) is bounded on the east by sparsely developed rows of hills and on the west by San Francisco Bay. EBDA's primary mission is to manage the construction and operation of a twenty-mile long interceptor which extends the length of its service area, plus an eight-foot diameter outfall which extends seven miles into the Bay. When this "Super Sewer" is completed in late 1980, four secondary treatment plants will discharge 50 million gallons of effluent through it.

Other than the two cities which are members of EBDA there are four others with active or potential interest in the reuse survey. Three of these are served by one of the member sanitary districts and potentially could have used reclaimed water for irrigating parks. The fourth has jurisdiction over the end of the outfall and is interest in minimizing the volume of effluent discharged. Three agencies supply water in EBDA's service area. One is a city, Hayward, which is a member of EBDA. The other two are special districts. One of these districts has jurisdiction over a ground water basin. These two districts could possibly have perceived reclamation by EBDA as competition. One was interested in protecting the ground water basin which it was recharging with imported supplies. There are a number of other local agencies with interest or concern in the program. The most important of these will be discussed below.

POLICY LEVEL COORDINATION

When EBDA decided to evaluate the feasibility of using reclaimed water in its service area a preliminary assessment was made of the constraints which had to be overcome. The most critical of these constraints were:

...Where could the reclaimed water be used and who would purchase it?
...What were the implications of public health regulations?
...What would the costs of supplying the reclaimed water be and against what costs would project feasibility be measured?
...What other public agencies and private groups could be supportive of, or concerned about EBDA's intention to proceed?

After having assessed these constraints procedures were developed for coming to grips with them. The first step was to provide a role for those agencies which had interest or concern in the study. EBDA elected to bring them together into a nine-member Reclamation Policy Advisory Committee (RPAC). The agencies selected and the individuals they appointed to represent them were: the two water districts; one designated a member of its board, the other, its chief of planning; the city into which the outfall discharged appointed a councilwoman; and an industrial association appointed two of its officers. To this group were appointed a private citizen who was interested in reclamation and two members of EBDA's governing commission. The responsibilities which were given to this committee were to advise the agency commission on the following matters:

1. Scope of work, level of effort, funding, review of alternatives and of recommendations of study reports.
2. Acceptability of alternative projects to other public agencies, to industry and to the general public.
3. Resolution of areas of overlapping responsibility with other public agencies.
4. Conduct of a public information program.
5. Recommend Authority policies.

The next step was to employ a consulting firm which could make the actual reuse survey and which had competance to provide the technical guidance necessary. The RPAC played a major role in selection of that firm. The firm chosen was Lowry and Associates from Irvine, California, which had had extensive experience in developing reclaimed water systems in that community. They began work in January 1978. Their assignment was to survey our service area to determine those sites where reclaimed water might possibly be used, to develop a screening procedure which would eliminate all non-viable projects and to perform the screening process to find those relatively few sites where reuse would be viable.

SCREENING PROCEDURES

Lowry people developed a ten-step screening procedure (which is diagramed on Figure 2). The first step was a brainstorming approach which listed all of the possible types of reuse that could be conceived. (These are shown on Table I). These included approximately 65 uses ranging from domestic supply to car washing and aquaculture. The second step was to screen out those uses which were not viable. The criteria used for that screening procedure were:

...State Department of Public Health regulations on use of reclaimed water as prescribed in Title 22 of the State Administrative Code. These specify bacterial limits and treatment levels for irrigation and for recreational impoundments.

...State Department of Health concerns other than those specifically mentioned in the Title 22 requirements.

...Potential enactment of future stricter Title 22 requirements.

...State Water Resources Control Board and Regional Water Quality Control Board requirements for reclaimed water use.

...Physical and chemical requirements of the potential users, e.g., physical clarity of effluent to prvent sprinkler clogging.

...Experience in other geographic locations on economics of treatemnt costs, dual distribution system, etc. for existing versus new developments.

Using these criteria the 65 uses were rated on the probability of their near-term acceptance, i.e., how realistic it appeared that each type of use could be implemental. Reference to Table I shows that these uses were divided into the following five categories (priorities):

Category 1 (First Priority) - Demonstrated economic feasibility or cost effectiveness and institutional viability.
Category 2 (Second Priority) - Institutionally viable with moderate cost effectiveness or demonstrated cost effectiveness but with

only moderate institutional viability.
Category 3 (Third Priority) - Both institutional problems for implementation and probable high costs.
Category 4 (Fourth Priority) - Very high costs and institutionally not acceptable at this time.
Category 5 (Lowest Priority) - Not viable without landmark technological breakthrough. May never be viable.

Only categories 1 and 2 appeared to be feasible after this screening. The four types of use in these two categories were:

1. Industrial
2. Landscape Irrigation
3. Agricultural Irrigation
4. Wildlife Enhancement/Recreation

Therefore all subsequent screening would be of these four types of use.

The next step in the survey was to make a physical survey of the 200 square mile service area to determine all possible sites, regardless of location or potential reuse volume, where these four types of use could occur. This survey was made from maps, by reviewing planning department, water and sewerage records, chamber of commerce publications, by interviews of agricultural extension people and by cruising the area. More than 1,000 such sites were found. These included parks, school grounds, cemeteries, farms, golf courses and industries.

It was apparent that all of these 1,000 uses could not feasibly be served. Many were small and located a distance from the source of reclaimed water. However, it was possible that many of these small individual uses could be grouped into viable clusters. Therefore, the third screening was designed to eliminate those sites which would not be economically served either individually or as part of a cluster. A sample screening demonstrated that a useage of 10 acre feet per year was a reasonable breakpoint. The next step was to verify that those remaining users could indeed use at least 10 acre feet per year. To do this all were interviewed by telephone. They were asked for information on the nature of their operation (or product), water demand and water quality requirements. They were also given an opportunity to express their wishes to use reclaimed water. This screening step permitted reduction of potential users to 350.

The next step required preliminary determination of the costs to provide service to these 350. In order to identify as efficiently as possible the locations which were too far away with too little demand, a screening technique was developed to categorize the specific users according to their distance and volume. A family of curves was developed (Figure 3) to facilitate calculation of the unit cost of reclaimed water service for the multitude of users. These curves show the relationship between the annual cost per acre foot of reclaimed water, the volume of demand, and distance between reusers, or between reusers and source of reclaimed water. The figure indicates the unit cost of reclaimed water if the demand and distance between reusers are known. (The unit cost estimated on these curves represents only the incremental cost of facilities to serve a user from its neighboring users.)

The criteria and assumptions that were used in developing these curves are:

1. Minimum treatment required above secondary would be filtration plus chlorination.

2. Peak flow rate = Annual Demand x 0.18 x 2.0/30 days (Peak Summer Month Rate delivered over 12 hour period.) for landscaping and agricultural reusers.
3. Pipe Selection Velocities - Min - 4 fps, Max - 7 fps.
4. Power costs - $.04/KWHR.
5. Updated pipeline cost estimates to ENR2900 (Projected 20 cities index for January 1979.) Capital Recovery Factor 40 years @ 7%.
6. Capital and O&M Annual Cost = $20/af/yr for filtration plus chlorination. Capital Recovery Factor 20 years @ 7%.
7. Pump Station - Annual Capital and O&M Cost = $15/af/yr.

When estimating the unit cost of reclaimed water service of reusers, distance between two reusers, or between reuser and source of reclaimed water was first measured. Depending upon the reclaimed water demand, the unit cost of reclaimed water was found on the graph. Then reusers were grouped into categories according to unit costs:

Category 1 - reusers with unit cost of less or equal to $100/af;
Category 2 - with unit cost of $100-$200/af;
Category 3 - with unit cost of $200-$350/af;
Category 4 - users with unit cost from $350-500/af; and
Category 5 - above $500/af.

Category 4 and 5 users were eliminated during this screening process. The logic was that if a reuser could not support the facility cost from its closest neighboring user, it certainly could not support the additional prorated share of facility cost from the source of reclaimed water. Further, the cut-off at $250/af is related to cost of domestic service. This procedure permitted reduction of the number of sites to be investigated in more detail down to 140.

The next step was to eliminate those sites to which distribution costs would be inordinately high. Pipeline construction costs were the main criteria here. Distribution networks were laid out for each of the 140 surviving sites and the cost of pipelines estimated to convey water from the closest neighboring user. A cut-off point of $200/af was used for this screening. The logic was that if pipeline cost alone were more than $200/af of reclaimed water, then when filtration and pumping facilities and operation and maintenance costs are added, it will not be cost effective. Further, at this level of system layout, almost every project involved some form of landscape irrigation reuse, which would involve a minimum of filtration for additional treatment.

This final screening in the survey reduced the number of potential projects to eighteen (at 20 sites) which appeared to be viable. These eighteen and the estimated volume of reclaimed water they could use included:

REUSE SITE	EST. REUSE VOLUME ACRE FEET/YEAR
Six industries	614
Three farms	2,050
Two golf courses	866
Six parks and school grounds	2,248
Three marshes	7,500
TOTAL	13,278

TECHNICAL COORDINATION

In proceeding through this ten-step process it was necessary to incrementally make technical assumptions and decisions. The broader implications of many of these decisions made it wise to organize a technical advisory committee to serve as a sounding and review body for the consultant. Therefore, EBDA invited its five member agencies and the two water districts in its service area to designate representatives to sit on such a committee. Each designated the agency engineer-manager or a senior level-engineer. That technical committee met monthly during the conduct of the survey. As a group they provided concise technical guidance to EBDA staff and consultants. As individuals they served as excellent sources of pertinent technical information.

To achieve most effective use of the two committees, i.e., policy and technical, and of the consultant's time, both committees were convened on the same afternoon once a month. The technical committee met first. Several members of the technical commitee would then sit in with the policy committee. Technical matters could then be presented in a succinct manner for policy committee consideration.

INSTITUTIONAL COORDINATION

These committee meetings also provided opportunities for coordination of the survey with other interested agencies. Meeting notices and minutes were distributed to such agencies as the State Water Resources Control Board, the State and County Health Departments, other cities and a park district within our service area plus county flood control and mosquito abatement districts. This committee mechanism provided an excellent administrative tool for coordinating with the State Water Resources Control Board's Office of Water Recycling (OWR). OWR is the reclamation agency established by the Governor. Among its responsibilities is the administrative guidance of studies such as EBDA's. OWR designated a project coordinator to work closely with EBDA. His participation with the two committees permitted him to play an active role in guiding the study. This was far more effective than the alternative procedure of presenting the finished survey document to OWR for their required approval. The OWR designee was party to those decisions which were important to his agency. There were, therefore, no great differences in his and EBDA's perceptions of the survey results.

SUMMARY AND CONCLUSIONS

EBDA has recently completed a survey of its service area to determine where reclaimed water could feasibly be used. Although many wastewater management agencies are performing such surveys it is believed that EBDA's has several unique features which will provide insights for similar surveys by others. These unique features include:

...The availability of a source of reclaimed water extending the full length of EBDA's service area.

...The geographical area covered by the survey ranges from being completely urbanized on the northern to rapidly urbanizing on the southern end.

...The survey was a finer-grained search for reuses in this urbanized/urbanizing area than has been made by others.

...There are many local agencies within the survey area which have some jurisdiction over use of reclaimed water.

In addition to these features there are several others which act as constraints to reuse. These are:

...Fresh water supplies are of good quality, reasonably priced, and adequate for the areas' needs through the year 2000.

...Public health regulations are sufficiently stringent to result in pricing certain reuses, e.g., irrigation of parks and playgrounds, out of the market. These regulations also effectively preclude use of reclaimed water for ground water recharge.

...Ground waters being used for community supplies must be protected from infiltration, thus reducing the areas where reclaimed water can be used for landscape and agricultural irrigation.

The study, therefore, had to be designed to accommodate these unique features and constraints.

In performing the survey hands-on experience was gained on steps which will maximize the effectiveness of such surveys. In general, early planning assumptions were reenforced and refined. Specifically the approach to early planning for the survey was reenforced. The most important steps in that planning were the preliminary assessment of constraints, issues and interests plus the development of a mechanism for the active participation of other interested parties in the survey. These two steps permitted a reasonably thorough scoping out of the technical and institutional aspects of the survey. They also permitted resolution of issues without polarization of positions.

We also found that the fine-grained survey did adequately respond to those questions about other uses. When the survey was presented to the public at two workshops, instead of lists of uses overlooked, compliments were received on its thoroughness.

Two of the technical items which would improve survey effectiveness are: 1) Zeroing in early on larger reclaimed water uses and 2) ten acre feet per year is a reasonable lower limit for screening out the smaller, uneconomic uses. Application of these factors would permit more intensive evaluation of the feasibility of larger uses.

It is believed that the survey was cost effective. The total cost, including an environmental impact assessment and administrative expenses, was $170,000. If this was allocated by unit land area or population in the survey area it would be $850 per square mile or $0.38 per person. This is a reasonable price for the level of credibility the survey has achieved among other public agencies and with the general public.

The survey took nine months (January through September 1978). For those resulting projects which appear to be viable EBDA is preparing a project report under State and EPA grant guidelines and an Environmental Impact Report under State guidelines. In parallel with these engineering and environmental evaluations, EBDA is conducting an administrative program which consists of:

1. Determination of how to apportion reclamation management responsibilities within the EBDA organization.
2. Determination of specifically how to interface with the water agencies.
3. Requesting preliminary commitments from potential reusers on their intentions to purchase reclaimed water.

By September 1979, the engineering and environmental analyses will have been completed. By January 1980, management and pricing arrangements should have been sufficiently completed to permit the start of facilities design.

TABLE 1

PRELIMINARY ANALYSIS OF RECLAIMED WATER USES

EBDA REUSE STUDY

	Use	Probability of Acceptance	Probable Level of Treatment	Remarks
I	MUNICIPAL/DOMESTIC			
A.	Direct Domestic Consumption	5	-	SDH will not permit direct consumption regardless of treatment used.
B	Toilet Flushing			
	Residential	4	-	SDH can be expected to strongly resist allowing R-W into residential building
	Public (Schools, etc.)	4	-	SDH can be expected to strongly resist allowing R-W into hospitals, schools, and similar public buildings
	Industrial and Institutional	3	S_B, F	May be possible, but treatment and monitoring levels could be unreasonably high
C	Landscape Irrigation and Restricted Lakes[a]			
	Parks and Rec Fields	2	S_B, F, Maybe C	Acceptable use if treatment is adequate
	Multi-Family and Common Areas	2	S_B, F, Maybe C	
	Playgrounds and School yards	2	S_B, F, Maybe C	
	Public Buildings	2	S_A, Maybe S_B with F	
	Industrial Areas	2	S_A, Maybe S_B with F	
	Single-Family Lots	4	-	SDH can be expected to strongly resist allowing R-W to be used at individual homes and handled by home owners
	Hotel Grounds	2	S_B, F, Maybe C	Depends on location - SDH will not allow near patios and areas where people congregate and where food is served
	Commercial Areas	2	S_B, F	
	Golf Courses	1	S_A	Excellent use
	Green Belts, Cemeteries, Slopes and Medians	1	S_A	Excellent use
D	Street Cleaning (Sweepers)	3	S_B, F	Quantity used will be small and distribution system will be extensive treatment must reflect degree of exposure
E	Fire Protection			
	Schools and Hospitals	3-4	S_B	May be acceptable where dual public distribution systems can be constructed. Will be resisted for indoor sprinkler systems, particularly in buildings where the public gathers and where food is served. Quantities used will not offset cost of system unless fire and irrigation supplies or fire and industrial process supplies can be put in one system and domestic supply put in another system
	Residential Areas	4	S_B, F	
	Industrial	2	S_A	
	Public and Shopping	3	S_B	
F	Air Conditioning[b]			
	Single Family	5	-	Consumption would be very low - would be resisted by SDH
	Industrial	2	S_A	Good use if salt build-up is not excessive
	Schools and Hospitals	4	-	May be permitted if use limited to certain areas
	Commercial and public	3	S_B	Acceptable if used in large control systems
G	Land Fill Operations	2	S_B	Good use, but consumption may be small - runoff and leaching must be considered
H	Vehicle Washing			
	Commercial and Service Stations	4	S_B, F	Acceptance will be influenced by who handles the R-W and how much exposure employees and public will have. Where employees are public and where automated equipment is used as in large industrial and commercial operations, use will be acceptable if R-W is adequately disinfected
	Public Agencies	2	S_B, F	
	Car Rental and Auto Dealers	3-4	S_B, F	
	Trucks, Trains, Const. Equip., Farm Equip., etc.	2	S_B	
I	Decorative Fountains			
	Public and Commercial	3	S_B, F	Acceptance will depend on the amount of public contact and exposure. May be acceptable in residential and shopping areas with a very high level of treatment. A lower level of treatment may be acceptable in industrial areas and at other locations away from children and eating areas
	Industrial	3	S_B, F	
	Residential	4	-	
J	Swimming Pools			
	Community	4	-	Acceptance will be very difficult to obtain. Level of treatment required will be high. Consumption will be low. Liability will be a major concern
	Apartment and Hotel	5	-	
	Single Family	5	-	
K	Community Gardens			
	Public and Large Private	3	S_B, F Maybe C	Acceptance will depend on degree of control and whether or not food crops will be grown
	Individual	4		Probably not acceptable, particularly if food crops are grown and plots are small. High level of treatment will be required
L.	Sewage Treatment	1	S_A, Varies	Very acceptable. Should already be standard practice
M.	Land Use Planning	1-2	Varies	Potentially very acceptable to SDH. Depends on type of use attracted

TABLE 1 (Continued)

PRELIMINARY ANALYSIS OF RECLAIMED WATER USES

EBDA REUSE STUDY

(Continued)

		Use	Probability of Acceptance	Probable Level of Treatment	Remarks
II		GROUNDWATER MANAGEMENT			
	A	Recharge (Spreading)	3	S_B, F, Maybe N_R, C, A & D	SDH will be restrictive, but if conditions are right and water is short, it may be a good use
	B	Sea Water Barrier (Injection)	4	S_B, N_R, F, C, A, Maybe D	Approval from SDH will be difficult to obtain. Pilot work, a high level of treatment and correct conditions must be considered
III		INDUSTRIAL			
		Various individual process uses should be reviewed			
IV		CONSTRUCTION	2	S_A	Good use if project is near source of supply, otherwise, uses may be scattered and intermittent. If personnel will come in contact with R-W, a higher level of treatment will be required.
V		COMMERCIAL			
	A	Car Wash (See above Item I - H)			
	B	Retail Nursery	3	S_B, F	Acceptable if R-W properly applied, and kept away from food crops and the public. Level of treatment will depend upon level of exposure.
VI		AGRICULTURE			
	A	Feed and Fodder	2	S_A	Good to excellent uses as long as water quality is suitable and Title 22 requirements are met. Higher levels of treatment may be required in some instances, and mineral increases in underlying groundwater must be considered
	B	Pasture	2	S_A	
	C	Fiber and Seed	1	S_A	
	D	Washdown	2	S_A	
	E	Tree Farming	1	S_A	
	F	Commercial Nursery	2	S_A	
	G	Land Reclamation	2	S_A	
	H	Human Food	3	Title 22	Title 22 requirements must be met. In addition, metals may require removal. Studies are being conducted in Monterey to determine the effect of R-W on various crops. Results of those studies will have direct bearing on irrigation with R-W
	I	Hydroponics	4-5	-	Benefits and consumption from hydroponics are questionable. Quality must be high
VII		RECREATION			
	A	Lakes and Ponds Fishing and Boating	2-3	S_B, F, Maybe N_R and C	Acceptable if fish aren't eaten and there is no body contact. Phosphorous removal may be necessary (See Title 22)
		Swimming	4-5	-	Approval will be difficult to obtain. There are potential problems with liability. Nutrient removal necessary to maintain clarity
		Hunting	2	S_B, Maybe F and N_R	Acceptable use. Nutrient removal may be necessary
	B	Hiking and Horse Trails	2	S_B	Good use if irrigation practices preclude spraying R-W on people and there is no runoff to stream beds
VIII		WILDLIFE ENHANCEMENT			
	A	Marsh Enhancement	1	S_B, Maybe F	Excellent use. Marsh will remove nutrients. Large quantities can be used
	B	Irrigation of Feed Areas	2	S_B, F	Acceptable. Will be necessary to exclude public while irrigating
	C	Improvement for Game	2	S_S	Acceptable. Will be necessary to exclude public while irrigating
	D	Streamflow Enhancement	1	S_B, F, N_R, Maybe C	May be acceptable in limited areas. Potential for body contact requires high level of treatment. Effects on groundwater and receiving waters must be considered
IX		AQUACULTURE			
	A	Farming			
		Fish	4	-	R-W must be highly treated and fish may not be considered edible
		Shellfish	5	-	Shellfish are very susceptible to contamination
	B	Sport Fishing Ponds	4	-	Without very high levels of treatment and monitoring it is doubtful that fish could be taken or eaten
	C	Fish Hatcheries	4-5	-	Treatment level and monitoring would be extensive
	D	Brine Shrimp Production	4	S_B, F	Salinity of R-W is not adequate

TABLE 1 (Continued)

PRELIMINARY ANALYSIS OF RECLAIMED WATER USES

EBDA REUSE STUDY

(Continued)

Use	Probability of Acceptance	Probable Level of Treatment	Remarks
E. Algae Farming	4	-	Market for algae is limited
F. Aquarium Plant Farming	3	S_B, F	Very limited use - otherwise acceptable. Metals removal may be required

Probability that use will be acceptable

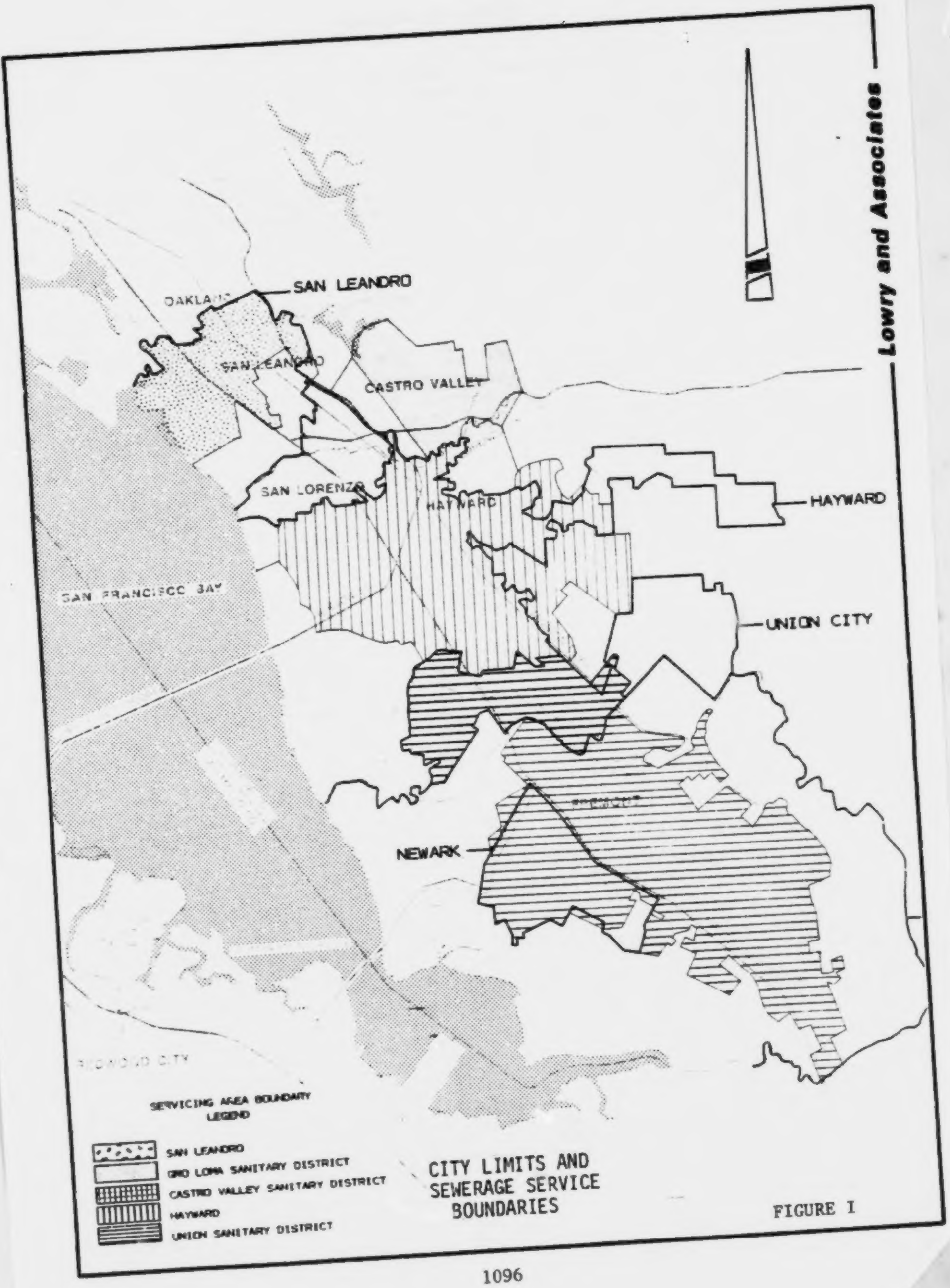
1 - Highest
2 - Medium High
3 - Medium Low
4 - Low

a) Impacts on groundwater must be considered

b) Buildup of salt concentration must be considered if evaporative cooling is anticipated

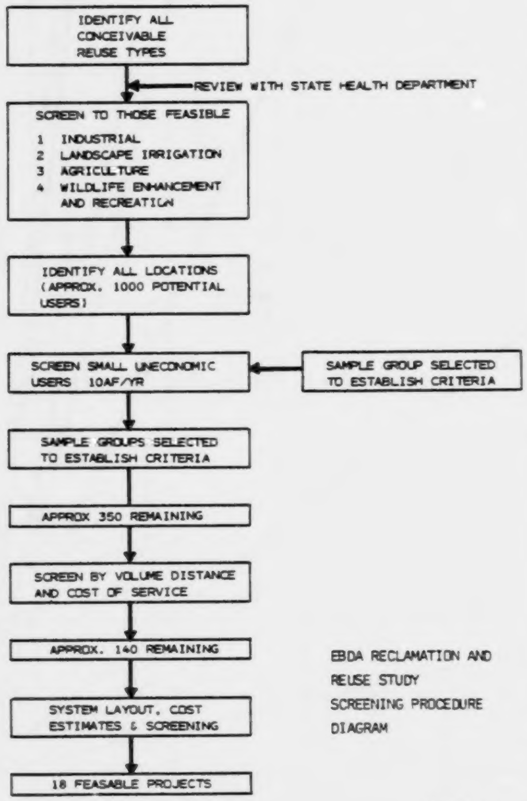
Treatment Level

S_A - Secondary and Disinfection (30/30/23)
S_B - Secondary and Disinfection (30/30/2.2)
F - Filtration (2 NTU)
N_R - Nitrogen Removal
C - Chemical Coagulation and Sedimentation or equivalent
A - Carbon Adsorption
D - Demineralization



CITY LIMITS AND
SEWERAGE SERVICE
BOUNDARIES

FIGURE I


IDENTIFY ALL CONCEIVABLE REUSE TYPES
REVIEW WITH STATE HEALTH DEPARTMENT
SCREEN TO THOSE FEASIBLE
1 INDUSTRIAL
2 LANDSCAPE IRRIGATION
3 AGRICULTURE
4 WILDLIFE ENHANCEMENT AND RECREATION
IDENTIFY ALL LOCATIONS (APPROX. 1000 POTENTIAL USERS)
SCREEN SMALL UNECONOMIC USERS 10AF/YR
SAMPLE GROUP SELECTED TO ESTABLISH CRITERIA
SAMPLE GROUPS SELECTED TO ESTABLISH CRITERIA
APPROX 350 REMAINING
SCREEN BY VOLUME DISTANCE AND COST OF SERVICE
APPROX. 140 REMAINING
SYSTEM LAYOUT, COST ESTIMATES & SCREENING
18 FEASABLE PROJECTS

EBDA RECLAMATION AND
REUSE STUDY
SCREENING PROCEDURE
DIAGRAM

FIGURE 2

Lowry and Associates

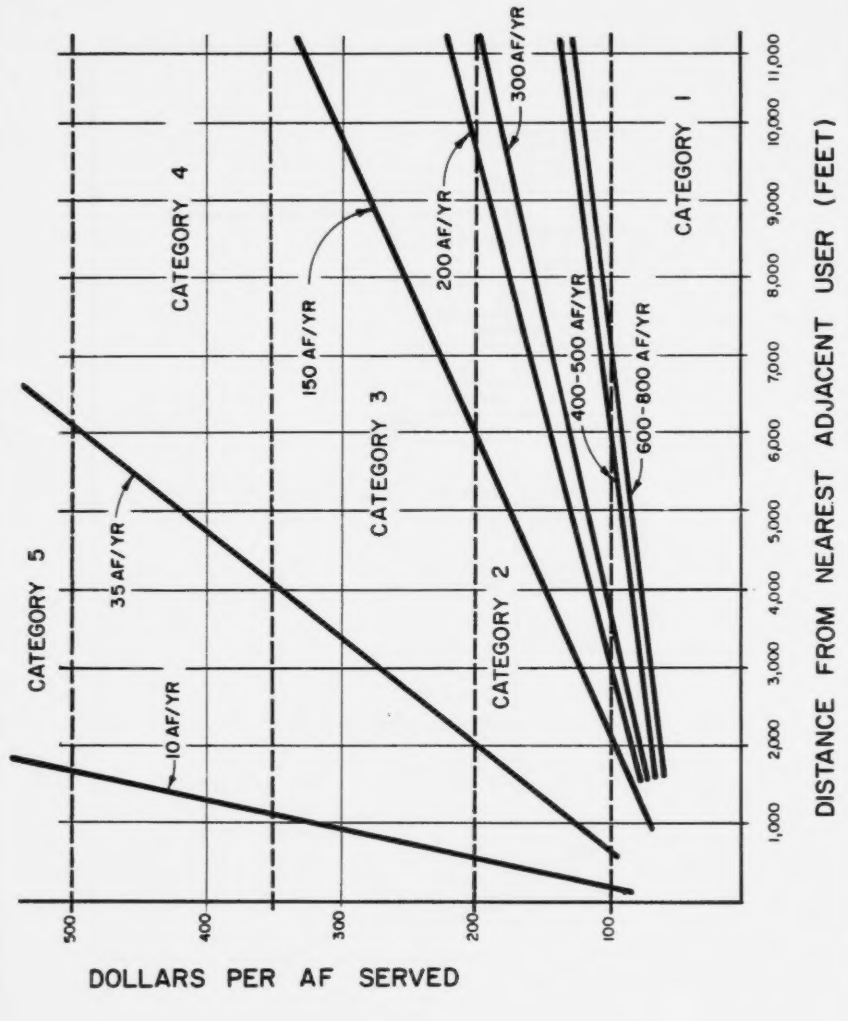

CATEGORY 5
CATEGORY 4
CATEGORY 3
CATEGORY 2
CATEGORY 1
10 AF/YR
35 AF/YR
150 AF/YR
200 AF/YR
300 AF/YR
400-500 AF/YR
600-800 AF/YR
500
400
300
200
100
DOLLARS PER AF SERVED
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000
DISTANCE FROM NEAREST ADJACENT USER (FEET)

FIGURE 3

INSTITUTIONAL BARRIERS TO WASTEWATER RECLAMATION AND REUSE

Richard L. Perrine
Professor of Engineering and Applied Science;
Chairman, Environmental Science and Engineering
University of California, Los Angeles

Water is a key determinant in the development of California. Reuse of reclaimed wastewater is technically feasible and, in some instances, economically attractive. Since the resource potentially available is some 25 percent of municipal and industrial consumption, it can augment supply and reduce problems and costs which might follow further development of remote fresh supplies. Despite this, reuse in California has grown slowly. A reason may be institutional barriers which must be addressed in order to implement the 1977 Policy and Action Plan for Water Reclamation in California (1), and to achieve the announced target of 400,000 acre-ft more per year of reclaimed water use by 1982 (2).

This paper summarizes an extensive report (3) which has sought to answer the question of why, despite benefits of reuse and increasingly available highly treated sewage effluent, there has been such a limited increase in reuse even in low-contact applications. The study has accepted increased reuse as justified by reduced energy consumption, recapture of investment in sewage treatment, replenishment of depleted aquifers, and preservation of fresh supplies for in-stream and other uses. The analysis identifies barriers to reuse and indicates how these may be overcome. Its foundation is case studies of projects located (as a necessary convenience) in southern California. The objective thus has been to identify and to analyze the institutional problems which arise when water resources professionals attempt to develop reclamation and reuse projects. An accompanying objective is to identify potential means of reducing some of these problems so that future projects are easier to review and to implement.

Following baseline water management information, the body of the paper divides conveniently into three major sections. The first of these summarizes analyses of issues in four areas which emerged as common and critical during initial stages of our project (3). They can be classified within broad groupings as administrative-institutional arrangements, legal issues, water pricing factors, and public health risk and standard-setting. The next section summarizes and relates to issue areas information gleaned from the three case studies, covering, respectively: San Jose Creek Water Reclamation Plant, Walnut Valley-Pomona Water Reclamation Plant, and Water Factory 21. The final major section summarizes our analyses by means of twelve major issues of concern, and notes recommendations which follow from the study.

From its initial formulation, the basic study (3) received support from state and local organizations concerned with wastewater reclamation and reuse, many of which nominated representatives who served on a Project Advisory Committee (representation is given in the Acknowledgements). This provided a broad, objective perspective for review of the regional study. The Advisory Committee provided important services at three critical points in conduct of the study: in its design, choice of case studies, and in reviewing results of the project.

WATER MANAGEMENT IN CALIFORNIA

California depends mightily on transfers from water-rich, northern areas and on imports to meet supply needs. While most use is for agriculture, about 6 MAF/yr ($7.4x10^9m^3$/yr) is for municipal and industrial purposes, discharging about 3.1 MAF/yr ($3.8x10^9m^3$/yr) of wastewater. By the year 2000 wastewater may reach 4.7 MAF/yr ($5.8x10^9m^3$/yr) (4), providing reuse potential up to about 2.6 MAF/yr ($3.2x10^9m^3$/yr) (5). Geographical distribution represents a nonuniform extreme; 75 percent is centered in two densely urbanized coastal portions of the state, and other sources are scattered (6). (Reuse through large agricultural return flows and percolation to groundwater is not considered in this accounting.)

Within the South Coast Hydrologic Area of California, our basic study region, about 1.3 MAF ($1.6x10^9m^3$) of wastewater received primary or secondary treatment in 1973. Of this 83 percent was discharged to saline waters, ten percent was incidentally reclaimed, and only six percent was planned for reclamation and beneficial use (6). By the year 2000 this region could be reclaiming 700,000 AF ($8.6x10^8m^3$) more wastewater, equivalent to projected increase in demand over some twenty years.

From a more narrowly targeted viewpoint than the entire South Coast Hydrologic Study Area, 86 percent of water use in Orange and Los Angeles Counties lies within municipal and industrial categories, and only five percent is for agriculture (7). Of nearly 2 MAF/yr ($2.5x10^9m^3$/yr) furnished about 1.1 MAF/yr ($1.4x10^9m^3$/yr) enters sewers. Half could be reclaimed, though only about ten percent is reused at present. Historically, two types of systems have served the area: large coastal primary facilities discharging into the ocean, and smaller inland secondary facilities providing downstream hydraulic relief and reuse on an opportunity basis. Effluent quality can be a reuse constraint. A primary concern is total dissolved solids since salt buildup cannot be allowed to continue through repeat cycles. Specific contents also may be limiting. Thus it is fortunate that upgrading of coastal plants to secondary and inland plants to tertiary treatment has recently taken place. Currently 50,000 AFY ($6.2x10^7m^3$/yr) of the 127,500 ($1.57x10^8m^3$/yr) AFY of tertiary-treated flow is reused (7). The remainder is released to the ocean. By 1983, another 75-100,000 AFY ($93\text{-}123x10^6m^3$/yr) in tertiary-treated wastewater probably will meet legal discharge requirements.

Though the area may have contractual entitlements to meet net water demands through the turn of this century, legal and political uncertainties plus escalating energy costs may make imports questionable. Since local surface and groundwater is developed, reclamation of urban wastewater may prove useful to meet supplemental demands.

ISSUE AREAS DERIVING FROM WASTEWATER RECLAMATION AND REUSE

Administrative-Institutional Arrangements

Administration of wastewater reclamation is guided by federal and state statutes and regulations, magnified in importance because several are tied to grant funding so essential to project initiation. Regulations are implemented through federal, state and local agencies, leading to issues which may become barriers to increased reuse within California. Issue or policy areas are not separate, nor mutually exclusive. Thus they must be examined in terms of both narrow and wide ranging implications for the total management of the state's water resources.

Legislation, Regulation and Administrative Procedure. Two federal laws provide a foundation for wastewater use (while serving many other needs): the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) together with the Act as amended in 1977, the Clean Water Act (PL 95-217), and the Safe Drinking Water Act of 1974 (PL 93-523). The Environmental Protection Agency, the primary federal water quality regulatory agency, has most influence in implementing legislation. It also holds the federal purse strings. Other agencies with some interest through their water supply and facility responsibilities are the U.S. Bureau of Reclamation and the Army Corps of Engineers.

California laws and policy derive from the California Water Code, primarily the Porter-Cologne Water Quality Act of 1969 (8). Other policy instruments are the Wastewater Reclamation Criteria of the State Department of Health Services (DOHS) (9), the Policy and Action Plan of the State Water Resources Control Board (WRCB) (1), and the California Water Plan of the State Department of Water Resources (DWR) (5). Executive Order B-36-77 issued October 13, 1977 by Governor Edmund G. Brown, Jr., established the Office of Water Recycling (OWR) within the State WRCB "to make available an additional 400,000 acre-feet of reclaimed water by 1982" (2). Primary responsibility for supply (DWR), for water quality (WRCB), and for protection of the public's health (DOHS) lead to shared but different state level responsibilities for implementation of reuse projects and enforcement of legislation. Other agencies provide advisory support as demanded by nature of the application (e.g., Department of Fish and Game for streamflow augmentation).

Local agencies are essential to the water management and decision-making process. The Southern California Association of Governments is a regional council with professional staff and with planning functions, including regional "208" planning. They have been directed by the WRCB to evaluate options needed to implement reuse projects. The Metropolitan Water District (MWD) is the primary water wholesaler to municipalities, holding long-term contracts for water both from the Colorado River Aqueduct and the State Water Project. MWD must be concerned lest reuse diminish its ability to repay large sunk costs.

Several administrative procedures have been established to regulate reclamation projects. The most important areas are obtaining project permits and funding. The regional Water Quality Control Boards (WQCB) accept permit applications. Comments are made by other interested agencies, and a decision may be appealed to the State WRCB. Environmental Impact Reports generally are required by the California Environmental Quality Act. Draft preliminary standards and criteria for a proposed project are set by the regional WQCB. If the State DOHS has not set criteria for a reuse application, the regional WQCB has responsibility to act. Coordination by the state Office of Planning and Research now is required to speed decisions, through its Office of Permit Assistance. Priority for funding is given to projects essential to meet discharge requirements. Proposals must consider alternative technologies, searching for means that are cost-effective and have least environmental impact. A complete economic and financial analysis is essential.

Planning and Administration of State Water Resources: Supply and Quality. Historically, there have been two differently oriented approaches to water: one of water supply, the other of wastewater quality after use. Public policy routinely has addressed each separately. Dual philosophies have been able to coexist because organizations have aligned themselves with these orientations. The relationship has been serial: water supply concerns end with delivery ready for use; water quality concerns begin following use. Reclamation is a bridging practice requiring better coordination between supply and quality management.

There appears to be a lack of coordinating authority by any one agency, and there has been less than desirable cooperation among them. There is much overlap in agency missions, and there has been no final coordinating agency between the DWR and WRCB. Thus duplication has occurred in the development of water management plans and wastewater reclamation integration plans. There also are questions as to the proper federal role in managing waters of a state, and difficulty in addressing reclamation outside the context of the whole state water resource management system. In large part these questions appear to arise because reclamation has been considered as a separate process.

Within California the DWR promotes development of supply, largely reflecting concern with meeting agricultural needs, and municipal needs of southern California, via the State Water Project. Development of reuse could foster a conflict of interest were it to make difficult meeting bond obligations. On the other hand, the WRCB tends to view reclamation as a part of the water quality control system. Reclamation becomes valuable with large reuse supplies, and thus from urban areas in coastal regions. Major water utilization is inland, where large urban wastewater supplies are lacking.

Coordination at present depends on the relatively new Office of Water Recycling (OWR) which must now merge supply and quality interests. This requires a change from historic WRCB staff orientation toward water quality planning. Concerns added include how wastewater is to be distributed, marketed, and incorporated into the fresh water economic system. Effectiveness of OWR will increase with more of a supply orientation, aiding DWR in the shaping of a share of reuse supply policy.

In addition to lack of supply and quality coordination, water pricing within the state has no consistent basis. The administrative issue concerns failure of agencies to recognize subsidies and other cost-modifying policies for both reclaimed and freshwater projects and their effects on economic decisions. Much is history and at one time had a good purpose. Retention, however, distorts evaluation. Particularly now that water is in short supply, it inhibits curtailment of waste.

Authority to Implement and Regulate Wastewater Reuse. At present the WRCB has authority to investigate waste and unreasonable use, and is required to consider reuse in its evaluations. Permit conditions may be changed or a permit revoked if water is not used for beneficial purposes. This might seem to provide a strong promotional base for reuse. However, the courts have strictly interpreted authority holding that such regulations constitute "no more than a policy statement which leaves the ultimate adjudication and reasonableness to the judiciary" (10). At present, however, this may not be a significant barrier.

Health standard setting is another regulatory issue area. The DOHS is directed to determine standards and criteria for various applications. In part they have done so, but have tended towards "failsafe" recommendations. Regional WQCBs and the State WRCB have the final authority in incorporating these standards into regulations. The Boards may set their own standards after conferring with the DOHS, though this might well be politically difficult. A possibility considered is to place standard-setting functions within the WRCB. This, however, also could create concern for potential conflict of interest; development of reuse at the expense of health protection. It also has been suggested that the DOHS only concern itself with the risks involved, and leave all inference or definition of acceptable risk to a broader-based decision-making body. Such a body would require membership lending both technical competence and political insight, and concern for all of health, economic and environmental impacts. The OWR currently plans to attempt to do this through development of a joint Health Advisory Panel (11).

Legal Constraints on Wastewater Reuse

The framework of laws covering reuse has been noted earlier, with other concerns related to administrative and institutional arrangements. This section discusses several of the more important legal issues perceived as constraints to increased wastewater reclamation and use.

Sale of Reclaimed Water. Under the Sewer Revenue Bond Law (12), wastewater treated by publicly-owned facilities may be sold for reuse. Many districts can own and operate sewer systems, and thus may sell collected wastewater. However, the right may be challenged under certain circumstances, as in California v. City of Roseville (13). In this instance the WRCB challenged a sale agreement contending that the water, which historically had been discharged outside the city and there appropriated by others, was "abandoned" water, and thus not available for sale. Unless an intent to relinquish possession was established, however, the water right of the reclaimer would appear protected.

Another issue relating to sales is whether or not an owner of a sewage treatment plant may sell within his service area water previously discharged to a stream. Downstream users may claim a right to continued supply of treated effluent because they have relied on its use. In the Stevens v. Oakdale Irrigation District case, the court held discharge of artificial flow during formative years of a district to not constitute abandonment of a future right to such waters (14). Reasoning in this case would seem equally applicable to the sale of reclaimed water by the owner of a plant previously discharging to a stream.

Antiparalleling Statutes. Water companies must be compensated for reduction in value of property because of duplication of facilities (15). Private utilities must obtain a Public Utilities Commission (PUC) certificate of convenience and necessity, and are regulated in all phases of operation. Water services must be provided in a service area. Often this means losses over many years with the expectation that future income will provide repayment. Publicly owned districts are political subdivisions not subject to the PUC. They may extend service to compete with private utilities, and they also have tax and other advantages.

If a political subdivision duplicates facilities of another unit, it must expect to pay just compensation. But utilitiy valuation is not an exact science. Sales are too infrequent to form a guide, and regulated earnings cannot be compared with income from other businesses. In addition, facilities often will not be made valueless. Thus, dependent on the circumstances of the specific case, antiparalleling statutes may impose a significant reuse barrier.

Public Health Standards. As noted in earlier sections, state agencies are directed by law to regulate wastewater reclamation. The DOHS is to establish criteria for each type of use which will protect public health and safety; regional WQCBs review applications and specify reclamation requirements based on criteria. Criteria have been established for land application of wastewater, industrial process use, and various recreational impoundment uses, but not for groundwater recharge. While a few groundwater recharge projects are ongoing in California, a virtual moratorium exists on expansion due to stringent standards that have been set. A Draft Memorandum of Understanding between the WRCB and the DOHS (currently tabled) was prepared stating that the regional WQCBs would issue requirements and award grants only for recharge projects that the DOHS first determined are capable of protecting health. Each project would be reviewed on a case by case basis.

The principal health concern from use of reclaimed water for recharge by surface spreading is the possible transport of stable organics through the aquifer, and their entry into community supplies when groundwater is recovered for use. Presence of stable organics may lead to possible carcinogens in a water supply and direct or indirect exposure. Thus while the potential for recharge use is large, until more definitive information is obtained DOHS has recommended that recharge operations be held at historic levels. This position constitutes a major barrier to wastewater reuse in California.

Direct injection into aquifers (particularly to repel seawater at coastal sites) also has been used. The WRCB and regional WQCBs have established a non-degradation policy to protect existing water quality in underground basins, though some change may be allowed. Thus basin plan objectives must be met, and a primary constraint on any direct injection may be limits set on salinity of injected water. Another potential constraint may be the Safe Drinking Water Act as amended (16), which regulates underground injection.

In adopting regulations, the DOHS must follow the California Administrative Practices Act which sets minimum procedural requirements (17). Setting reclamation criteria involves either decisions based on imperfect data or postponing decisions until better data are at hand; a difficult choice. A primary objective of the Act is to ensure meaningful public participation. Compliance with minimum requirements as has been common, particularly holding hearings only after criteria have been drafted, can promote unilateral, inflexible rulemaking, and create a significant barrier to reuse.

Authority to Implement Reuse. Permits to appropriate water are obtained from the State WRCB. The process includes a requirement that permittees periodically report on the potential for their use of reclaimed water to meet needs (18). The Board retains continuing authority and permittees may be required to implement reclamation to replace all or part of the water allocated (19). These requirements have seldom been invoked (20). The Board also has an established basis for regulations to interpret reasonable beneficial use and to conserve water (21). however, authority has not been used to implement wastewater reuse.

Economics, Financing and Pricing of Reclamation in Southern California

Economic and financial considerations may prove crucial to any reuse project. Analysis begins with available supply and potential demand for reclaimed wastewater. A cost comparison of reclaimed and fresh sources is necessary, particularly since competition between these may have financial implications. Specification of the true function of reclaimed water in water resources management in southern California is required as government funding and hence policy are involved.

Available Supply and Potential Demand. Using the Orange-Los Angeles Counties area, a survey was made of existing and planned treatment facilities to determine potential wastewater sources (22). Secondary or higher quality effluent would be used. Los Angeles County is projected to have 737 mgd ($2.8x10^6 m^3$/day) available in 1985 compared with current reuse of 38.5 mgd ($1.5x10^5 m^3$/day), and Orange County 168.4 mgd ($6.4x10^5 m^3$/day) with unquantified reclamation at present. Maps would show that in most cases there are potential reuse sites within reasonable distances of treatment plant sites (22).

Los Angeles County may have a potential reuse demand of about 300 mgd ($1.1x10^6 m^3$/day) and Orange County 200 mgd ($7.6x10^5 m^3$/day) in 1985. These data can be summarized in the form of a preliminary surrogate demand curve for southern California, Figure 1, with distributional data given in Table 1 (22). Use will require a cost competitive with fresh supplies after such things as piping retrofit and added operating costs for scale control are figured in, plus some small added savings. Cost components are indicated in Table 2. The very high terminal values of required savings accrue from injection for seawater intrusion barrier use. For this use at present high quality standards, and thus demineralization, have been required. Creation of this data base incorporates a host of assumptions, of course (22, 23, 24).

It would be preferable to treat savings which appear necessary and likely consumption separately for each category of use, since different quality could satisfy demand in each case. However, the data are far too sparse to permit such detailed treatment. It also would be desirable to know what incentive beyond a break-even point would trigger reuse in each category, and how large a disincentive regulation might provide. There are no data for such estimates.

Marginal Costs of Fresh and Reclaimed Supply. Fair comparison requires delineation of all public and private costs of water supply (see Table 2). Comparable planning periods must be used in the analysis to achieve this; for comparison, amortization for freshwater projects may be 50 years, Clean Water Grant projects 20 years, and private investments 5 years in typical situations. A common (perhaps lifetime) basis should be established for this purpose. Since marginal costs are sought, any sunk cost should not be considered. This includes the under-utilized State Water Project. Obviously, in the long term aqueduct and distribution system costs must be taken into account. Future changes in power costs are expected to be substantial. Some existing contracts include unusually favorable long-term rates, as low as 3 to 6 mil/KWH (off-peak). New contracts will be at much higher rates, such as 20 to 129 mils/KWH (25), and future fresh supplies will use sources requiring more energy to deliver. For reuse, the DOHS has proposed added treatment (activated carbon) as a prerequisite to groundwater recharge via spreading. Salinity control prior to injection also will be costly.

Additional fresh water could be delivered to southern California soon. Marginal costs first would be limited to pumping. However, new conservation facilities soon would be needed. Table 3 gives projected costs (26). A modest average is \$158/AF (12.8¢/m^3). However, the highest cost facility, the marginal facility to be replaced by reclaimed water, is projected at \$250/AF (20.3¢/m^3). Projected marginal costs of new freshwater supply thus are listed in Table 4. Power costs are of major importance. Current trends point to the highest figures given as perhaps most reliable. A reasonable conclusion is that in the medium term, additional freshwater delivered from northern California to Los Angeles will cost $350 to $600 (28.4 to 48.6¢/m^3) per acre-foot, probably trending toward the higher value.

Marginal cost estimates for reclaimed water can be developed in similar fashion. Most costs come from treatment, tailored to use, and distribution, usually site-specific. Costs of various unit processes are given in Table 5 (26). Secondary treatment and disinfection normally are required by regulation and thus are sunk costs of pollution control, not attributable to reclamation. Projected costs for various applications are summarized in Table 6 (23). Costs also depend on locations of supply and use, together with special, on-site or internal conversion costs. These are not readily generalized. However, example costs for an attractive reuse opportunity investigated for the Walnut Valley Water District are summarized in Table 7 (27). Internal conversion costs and quality reliability costs are not included. Costs would appear competitive with future freshwater supply, but other opportunities might not share the same cost-saving advantages.

In summary, wastewater reclamation may or may not be cost-effective with respect to use of fresh water. Fresh supplies now benefit from past subsidies. It may well be that the subsidy of Clean Water Grant funding is essential to make reclaimed water use a reality.

Financial Considerations. Financing of reuse generally combines federal, state and local support. The major source is the Clean Water Grant program, derived from PL 92-500 and PL 95-217 (28, 29). Federal and state funding are administered through the WRCB. These can provide 75 percent federal financing, plus 10 percent possibly added for an innovative, "alternative technology" approach, and another 12 1/2 percent from state funds.

Proposed wastewater treatment systems range from designated Group I (primary purpose water pollution control, secondary purpose reclamation), to Group IV (primary purpose reclamation). Only Group I projects receive funds, but a project may be transferred "up" from IV to I. An evaluation of reclamation potential is required in every instance. A market assessment must establish water users' ability and desire for reuse. The project must be shown cost effective, considering both dollar and environmental and social costs. A two-fold analysis is required of all applicants; an economic analysis, looking at total monetary costs and benefits regardless of who pays; and a financial analysis of prices users will pay indicating who pays and who benefits.

Regional WQCBs establish their priority lists and forward these to the State WRCB. A project to be funded first must be found to be fundable, then technically sound, then implementable, and to have a market. Reclamation projects within Group I must further show that the proposed system is more cost-effective than other disposal. Within Group IV incremental costs of reclamation facilities beyond disposal requirements must be more cost-effective than added fresh supply.

Local financing can be by ad valorem taxes and by revenue bonds. In many cases legislation permits issuance of revenue bonds without an election. An example is the Metropolitan Water District (MWD) requiring only approval by two-thirds of the governing board for bonds to be issued (30). If bonds were to rely solely on revenues from reclamation, there could be questions as to whether they would be accorded the same confidence as bonds secured by freshwater sales.

The most striking differences between fresh and reclaimed water are those of quality and reliability. Poorer reclaimed source quality may limit use; reliability of delivery also may be poor. For example, the city of Pomona contract reliability is 50 percent; service is interruptable half the time. For many uses this necessitates a dual hookup.

Another concern related to cost and financing is the possible need to compensate fresh water purveyors. Giant projects which have delivered water to southern California are repayed under contracts as lengthy as 75 years. Debt incurred must be paid. If reuse sales impact a freshwater purveyor such as the MWD, some restitution or transfer may be necessary. With careful management, it would appear that all potential supply likely will be needed, and all debt could be repaid.

Pricing of Reclaimed Wastewater. The economic/engineering development of water rates, as outlined by the American Water Works Association in their published booklet, involves determination of revenue requirements, allocation of cost of service to cost functions, and design of rate schedules to recover service costs from each class of users. Three cost functions are identified: administrative costs, commodity costs, and demand or peaking costs. Other types of rates include decreasing, constant and increasing block rates; tail block rates; and lifeline rates. Another pricing element is differential rates by use; different for agriculture than for domestic use, for example. Historically, agriculture has received low prices based on such arguments as perceived need to maintain a viable industry, and environmental enhancement through open space. Reduced prices for groundwater replenishment similarly may be said to provide benefits of water storage for future use.

Ad valorem taxes used to support water supply collect from those who indirectly derive benefits, and reduce revenue needed from sales; lowering prices. Established systems should need little of such assistance, which also often will be subject to consumer criticism.

The Walnut Valley District's project provides an example of how projects may be implemented (23, 27). Interviews established that a \$40 per acre-foot (3.2¢/m^3) savings would lure customers to reclaimed water, so this was guaranteed in order to get firm commitments. The Clean Water Grant Program 87.5 percent funding made this step feasible. Thus flexibility available to reclaimed water purveyors due to grant funding could allow coordination of pricing to meet that of freshwater. If public policy benefits from reuse where it is possible and reservation of fresh supplies for use where truly needed, prices can be adjusted so that this goal is achieved. The proper rate structure for freshwater and reclaimed wastewater could ensure the best use of these resources at the least economic and environmental cost.

Public Health Risk

Left untreated, wastewater contents would constitute a serious health risk. Treated, reclaimed wastewater imposes risks reduced at worst to low levels, though risk never can be totally eliminated. Thus an issue of major concern is the degree of health risk associated with various reuse applications. Adverse impacts on the natural environment also are of concern, usually protected against by the steps used to preclude harm to humans.

Risk must be placed in perspective. Public health risk assessment involves two steps: (1) measuring risk -- the probability and severity of exposure; and (2) judging acceptable levels of risk. The first is scientific, the second entails an inherently political element. To establish risk we must accomplish several tasks: (1) identify potential hazards, their magnitudes and sources; (2) determine potential release modes; (3) determine pathways through the environment; and (4) establish potential health and environmental effects. All are important to reuse decisions. Factors such as specific quality of reclaimed water, modes of reuse application, and site dependent factors all combine to present a complex web of relationships along which a decision as to which option to choose must be made. Potential pathways to the environment can be direct (air, water), or indirect (food chains, bioaccumulation). For planned reuse the primary pathways by which contaminants find their way back to man are by surface waters and groundwaters. Current instrumentation permits measurement at truly trace levels. However, the complexity of the system is such that this may be of little help; we not only need to know that something is present, but where it came from, over what time period, and what can be done to halt further contamination.

Sources of Risk. Extensive review of the literature (31, 32) indicates something approaching a concensus. There is agreement that the main contaminants of potential concern are viruses, heavy metals, and toxic (or potentially toxic) organic chemicals. Other contaminants also may be of concern, but less so for most applications. Of these others, bacteria, particularly indicator organisms, are very important as they may warn of the possible presence of less easily identified contaminants. Also, spray irrigation may distribute bacteria as aerosols. Parasites, though they can effect humans, usually are of minor concern from properly treated wastewater. While nitrogen is an essential nutrient, excess nitrogen as nitrates is unhealthy. Any dissolved contents may be of concern where salt buildup could threaten basin water quality.

Many viruses are excreted in human feces. Most outbreaks of virus disease are infectious hepatitus. Viruses in sewage are variable in number, depending on many factors. Treatment does not totally eliminate these agents. A difficult problem is that they are very difficult to quantify at the low levels remaining. Epidemiology is very difficult as people may ingest virus, develop infection but no symptoms, become carriers and transmit disease. Use of numerous water sources magnifies difficulty with detection. Under some circumstances viruses remain viable for long periods. Because they attach to clay minerals in soil, proper mangement of reuse opportunities such as groundwater recharge *should* preclude their reappearance in water supplies. However, if there are fissures or other shortcut transport pathways, this is not certain.

Some heavy metals -- Cu, Zn, Cd, Pb, Ni, Co, Mo, As -- when introduced into soil via wastewater face an uncertain fate. Most heavy metals concentrate in sludge, but traces remain in wastewater. They accumulate in surface layers of soil, building up to quantities that are not natural. The eventual impact is not known. Uptake by plants is one possibility; movement to ground or surface water is another.

Organic chemicals are the newest and perhaps potentially most serious concern. A wide variety have been found in water. Some are carcinogenic; some chlorine reaction products mutagenic. Pesticides and herbicides are likely to be included. Techniques for removal at the source appear not totally effective, and need to be developed. A particular concern is that injected water may carry relatively stable organics during recharge or land application. They may appear later, unrecognized, as a source of risk within a drinking supply.

Public Health Safety: A Matter of Judgement. Public health safety requires input information from the risk measurement activity, and judgement of what level of risk is acceptable. "Acceptable" risk implies a relative measure; not intrinsic, measureable, absolute. Health safety has been of concern throughout the history of wastewater reuse. The trend has been toward uses with greater exposure; from crop irrigation, to landscape irrigation, recreational reuse, and groundwater recharge eventually augmenting drinking water supply. If all reuse supply could be economically used in low contact and therefore low risk applications, this probably should be done. However, it does not appear likely.

In California, water quality assurance rests with the WRCB and the DOHS. As noted previously, regulatory roles and authority are established. Criteria and standards are intended to limit levels of contaminants so that water will be "safe" for any specified reuse purpose. With a difference in interests between agencies, resource agencies fostering overall water resource development while health agencies seek lesser risk, a break came when a certain level of contact was proposed. DOHS has stated that it will recommend against direct discharge and direct injection of reclaimed water for near-term supply augmentation. Recharge by surface spreading may be evaluated on a case by case basis. This places a substantial brake on increased use of recharge.

Concern has been raised that DOHS policy represents a "zero-risk" approach. DOHS maintains that until definitive health data are available to reduce uncertainty the only prudent course is to set conservative policies. Unfortunately, complete and reliable health information, if it could be obtained, probably would require decades to acquire. Further, no water source is risk-free. The question that must be answered is, what is the relative risk? This must be considered from a global viewpoint.

Approaches to Decision-Making. A factor not to be overlooked in decisions is the perception of the public regarding reuse. Studies reflect the power of aroused public opinion (33, 34). If it is not properly assessed and if the public is not well informed, crucial votes on bond issues, etc. will not achieve approval.

The process by which future decisions will be made is critical. As noted, the scientific process of measuring risk must be followed by the political process of judging acceptable level of risk. Clearly, a participatory approach is essential. An example methodology has been developed and tested by Davos (35, 36). The assessment of risks is used as an input into the social decision-making process, and evaluated for its relationship to the "public interest". Research steps include (1) identification of the various public, special interest, and decision-making entities; (2) ascertaining preferences, values, and the trade-offs various publics are willing to make with each reuse option; (3) identification of the alliances that support alternative reuse options, and assignment of values to risks; (4) assessment of the consistency of preferences and of values for the various coalitions.

The results permit a best-informed decision to be made. However, decision-makers must deal with the interdependencies between public health objectives and the remainder of society's goals. Thus the decision framework must be capable of maintaining an anticipatory attitude, and assessing both scientific information and interest group value judgement. Accurate risk information, much highly technical, will be essential to reduce uncertainty.

WASTEWATER REUSE CASE STUDIES IN SOUTHERN CALIFORNIA

Three wastewater reuse case studies were developed to provide examples for study of actual or potential barriers to reuse. All are within the coastal region of southern California encompassed by the current Orange and Los Angeles Counties Water Reuse Study (7). Each addresses significant issues and draws conclusions. Of necessity the review of case studies in this paper is very brief. Detailed presentation and discussion constitutes a major portion of our full report (3).

San Jose Creek Water Reclamation Plant Case Study

The San Jose Creek Water Reclamation Plant (SJCWRP) is owned and operated by the Los Angeles County Sanitation Districts (LACSD). Reasons for selection as a case study are that principal use of treated wastewater is for groundwater recharge by surface spreading and constraints have limited reuse. Recharge suppliers are the MWD (fresh imported water), and LACSD (reclaimed water). Water users are the Central and West Basin Water Replenishment District (CWBWRD), and several smaller users. The Los Angeles County Flood Control District actually spreads the water. Surplus water is discharged to a flood control channel. The usual agencies, their advisors and public group representatives are involved; groundwater regulators are the Regional WRCB and the State DOHS.

The SJCWRP is located in the San Gabriel Valley in eastern Los Angeles County (Figure 2). Capacity is 42,000 AFY ($52x10^6m^3$/yr) with current influent 33,600 AFY ($41x10^6m^3$/yr). High quality secondary effluent goes by pipeline to spreading basins in the Montebello Forebay, recharge area for the Coastal Plain aquifers (Figure 3). Facilities currently are being upgraded to tertiary treatment. The CWBWRD is an active water conservation organization charged with replenishing groundwater supply. Sources for this purpose are listed in Table 8.

The extent to which proposed uses of SJCWRP water will be implemented depends on resolution of several problems. A regional reclamation study has been initiated that will address some of these; the Orange and Los Angeles Counties Water Reuse Study (7). Objectives include identification of reuse options, definition of technical, health risk and administrative problems, and obtaining solutions. The Study's organization includes local, state and federal agencies, and an advisory group has been established to communicate to all interested organizations.

Summary of Case Study Conclusions. The concensus of case study workshop participants was that of the five issue areas ranked, the most important questions for managed reuse of SJCWRP effluent related to public health and economics, followed by legal-contractual concerns. Reliability and administrative-institutional concerns were minor. A brief analysis of each issue area follows.

PUBLIC HEALTH: The limit for recharge set by the CWBWRD on recommendation of the DOHS -- the historic level limit -- is the principal health-related barrier to increased reuse. Contributing directly are the conservatism of the DOHS, the lack of epidemiological data, and the role of the Replenishment District as protectors of the groundwater basin.

LEGAL-CONTRACTUAL: The CWBWRD has effectively tied up all reclaimed effluent from the SJCWRP either through the historic limit clause barring uses that could harm groundwater, or through the exclusive rights clause in their supply contract which would apply to other uses. Thus a surplus goes unused. Current negotiations may modify this situation.

ECONOMIC: Competition between fresh water from the MWD (at $41/AF or 3.3¢/m^3) and reclaimed water ($4-7/AF or 0.3-0.6¢/m^3) is not seen as a barrier to reuse at present. If reclamation projects continue to grow, the MWD will likely require compensation for loss of sales. Also, equity considerations -- which taxpayers pay for reuse and which benefit -- will need to be addressed. Price setting is unsure and may initially slow development, but should diminish as a problem once the number of projects increases.

RELIABILITY: Reliability of supply appears not to be a problem, though there could be concern in a severe drought if fresh supplies were unavailable, and effluent quality dropped due to severe conservation efforts.

ADMINISTRATIVE-INSTITUTIONAL: Several institutional conflicts are inherent in this multi-agency activity. However, these appear minor except as they influence primary issue areas.

Several of the institutional barriers identified in this case study were raised earlier as generic concerns: public health standard setting, interagency conflicts, price determination and cost transfers, and goals of water management. The CWBWRD has been an excellent customer for treated wastewater. However, the District has exclusive rights and "wastes" surplus effluent, discharged to the ocean. This is an example of two barriers (public health and legal) reinforcing each other. Arrangements noted in this case study are likely to differ from others if only because one agency involved is unique: the CWBWRD is the only such unit formed. To protect groundwater quality it has adopted a quasi-regulatory role. Further, downstream groundwater extractions are controlled by court-imposed adjudication, overseen by Watermaster services provided by the DWR. It seems clear that optimal use of SJCWRP effluent will require a regional approach to reuse. The Orange and Los Angeles Water Reuse Study, which incorporates all agencies in this case study, may successfully address many of the barriers noted.

Walnut Valley-Pomona Case Study

The case study area is the area that can be served by reclaimed water from the Pomona Water Reclamation Plant (PWRP); a region in eastern Los Angeles County about five by fifteen miles and overlying the Puente Basin (Figure 4) (39). Sources of water are imports from the MWD, groundwater, and surface water. These are summarized in Table 9. Surface and groundwater already are being used to the maximum possible extent; the reduction in groundwater use shown in Table 9 was because excess nitrate required blending with imported water. Estimates of future amounts of reclaimed water available average 10 mgd (3.8x10^{4}m^3/day). The quantity actually available will depend on three possible actions: (1) possible diversion of water now entering Pomona from the Montclair area to the Chino Basin Municipal Water District (CBMWD) Regional Plant No. 1, (2) diversion of some First Street Trunk Sewer flow into the PWRP; and (3) diversion from the South Pomona area to a Chino Basin plant. Clearly, differences in elevation, boundary locations, and differences in development rate of areas have contributed to a patchwork of systems within which a number of changes could be instituted to improve efficiency. These would influence where and how reuse takes place.

The Walnut Valley Water District (WVWD) is planning a conveyance facility for reclaimed water from the PWRP to the City of Industry, about ten miles (40). A criterion for funding is an assurance that reclaimed water will be used. Potential users are listed in Table 10. Securing these commitments has been the major factor slowing this project. Users hesitate until cost, quantity, quality and reliability are known. Most users would consume large quantities and are accustomed to paying a low, "tail block" rate; in this case, about \$109/AF (8.8¢/m^3). Based on studies reclaimed water will be sold at least \$40 below this rate. (By comparison, Pomona currently pays \$8.47/AF (0.687¢/m^3) for reclaimed water it takes from the PWRP, a price which happens to represent twenty percent of the plant's O&M costs.)

The City of Industry is interested in reclaimed water both for City use and by businesses within the city. Toward assuring a reclaimed supply, the City has purchased a ranch (outside City limits) as a reservoir site. The intended supply source is the San Jose Creek plant (SJCWRP). So far, no formal negotiations have taken place.

The largest project under consideration has been a proposal by the Pomona Valley Municipal Water District (PVMWD) to utilize about 22,000 AFY ($27 \times 10^6 m^3/yr$) from the entire PWRP flow plus expansion of the SJCWRP as well. Storage would be provided by reservoirs and use of the Spadra Basin Aquifer. Delivery would require use of current and added supply lines. An application for grant funds has been submitted.

Summary of Case Study Conclusions. Feasibility of wastewater reclamation in this case study area is without question. Agencies are searching for supplies. The challenge thus is to find economically attractive options, and appropriate, equitable means to distribute costs and benefits. A brief analysis of each issue area is as follows.

PUBLIC HEALTH: While there is some risk associated with reuse, in this case study area risk has been minimized by limiting most use to a mix of agricultural and landscape irrigation, and industrial use. Discharge to San Jose Creek of effluent not reused provides for primary contact recreation as well as incidental recharge, and subsequent use as drinking water. Thus greater use of reclaimed wastewater most likely will reduce the actual public health risk for population in the area.

LEGAL-CONTRACTUAL: Antiparalleling appears not to be a significant barrier in this case study. Two factors seem important. In this inland, water-short area, long-standing and good working relationships have developed. Also, (next section), economic concerns appear minor.

ECONOMIC: In this area there appears little fear that fixed costs cannot be recovered and properly distributed. Also, developers have paid much of the WVWD's capital costs, so purveyor's cost repayment needs are not large. The District's primary concern is variable costs. Grant funding for projects appears to make this achievable. Of course, mandated tertiary treatment makes the reclaimed product more useful and saleable, and provides a major share of project funding through Clean Water Grants.

RELIABILITY: Water supply is an important constraint in this area. "Connected capacity" in the PVMWD could serve five times the area's population. Purveyors know that Colorado River supplies will soon be reduced, and that the Peripheral Canal is not yet a dependable conduit from the north. These concerns appear to outweigh other factors in the thinking of potential reclaimed water suppliers and users.

ADMINISTRATIVE-INSTITUTIONAL: The most important issue in this issue area is that of grant administration. Several case study workshop participants indicated a preference to do without grant funding, if at all possible, to eliminate the reviews, delays, and "strings" attached to the processing of grants.

Despite a pro-development attitude throughout much of the region and an obvious desire to create more economic activity, participants uniformly denied any growth-inducing impact of reclaimed water availability. They saw their service as providing for demand already in existence. In this regard, it is interesting to note the words which form a portion of the WVWD logo: "Where Water Flows the City Grows".

Water Factory 21 Case Study

The area near the mouth of the Santa Ana River has provided a highly reliable source of artesian groundwater. Early in the century as agricultural demand grew, levels dropped and salt water intrusion began. In 1933, following legislation, the Orange County Water District (OCWD) began management of the groundwater basin. Since 1949 surplus imported waters have been spread to replenish the basin. As population grew rapidly in the 1960's and fresh supplies could be foreseen as allocated elsewhere, a project to inject tertiary treated wastewater was proposed and constructed by the OCWD: Water Factory 21 (WF 21) (41, 42).

Sea Water intrusion which would result from overdraft is repelled by a hydraulic barrier along a three mile section of aquifers (Figure 5). Seven extraction wells intercept brackish water just inland, to flow back to the ocean. A series of 23 inland wells inject through several strata into a freshwater mound further inland (Figure 6). WF 21 is a unique advanced treatment facility capable of reclaiming 15 mgd ($5.7 \times 10^4 m^3$/day) to meet all U.S. Public Health Service drinking water standards. Plant process flow is shown in Figure 7. To reduce dissolved solids (TDS) to a level meeting regional WQCB standards, a 5 mgd ($1.9 \times 10^4 m^3$/day) reverse osmosis facility removes 90 percent of TDS (to 50 mg/l). Reclaimed water is blended with connate water at 3,000 ppm TDS from deep wells to provide water for injection. A total of 23,000 AFY ($28 \times 10^6 m^3$/yr) of blended water are injected by the project; almost ten percent of total groundwater production in the OCWD (43).

Summary of Case Study Conclusions. The facility which is the central focus of this case study is unique; an example for possible future systems. Thus issues are expected to be different for this case study.

PUBLIC HEALTH: WF 21 must meet discharge requirements to conform to a Basin Plan. The primary constraint is the 550 mg/l TDS limit imposed on injected water to prevent groundwater degradation. Water from reverse osmosis is blended (50 mg/l) with ordinary tertiary treated effluent (1100 mg/l) to yield a combined flow at about 700 mg/l TDS. This water is blended with an at least equal amount from a fresh supply prior to recharge; a requirement of the DOHS. There is no apparent restriction on quality of this blending water; a "fresh" rather than reclaimed source. This is considered by some imposition of "dual standards" for surface and reclaimed water which in this instance works against reclamation. As noted earlier, the DOHS has not established criteria for groundwater recharge. The feeling expressed by case study workshop participants regarding lack of criteria could be summed up as "no goal, no action".

LEGAL-CONTRACTUAL: No important legal barriers were encountered in starting up WF 21. Orange County does not have adjudicated water rights. Any property owner can sink a well if he files for permits, pays an OCWD "pump tax", and does not export water from the District.

ECONOMIC: Present operating costs for the system providing blended injection water lead to a cost of \$148/AF (12.0¢/m^3). The average cost of water from deep wells is \$30/AF (2.4¢/m^3), from reverse osmosis \$166/AF (13.5¢/m^3), and advanced wastewater treatment \$147/AF (11.9¢/m^3). This cost is a factor of three greater than MWD replenishment water at \$41/AF (3.3¢/m^3). While costs seem high, energy needs of alternatives may in future raise true costs substantially (44). Much of the price differential accrues from cheap power rates for State Water Project water contracted for in the past.

RELIABILITY: No reliability concerns of importance were encountered in this case study.

ADMINISTRATIVE-INSTITUTIONAL: A group of agencies are principals in the operation of WF 21 and implementation of reuse. Strong political support was initially generated for the project, in part perhaps due to historical timing and its place as the first such reclamation project in California. No barriers were perceived during case study evaluation.

Operation of WF 21 has demonstrated technical feasibility of reclamation to meet drinking water standards, and also that a hydraulic barrier can be effectively operated. However, strong economic support would be essential to attempt development of another, similar project.

A Regional Overview

The case studies indicate that at present a project often proceeds with interests of a single agency or locality in mind. This may not assure the larger region of greatest reuse benefits. The currently most common route for implementation starts with project identification by a local agency, and an application for reclamation requirements to the regional WQCB. Here DOHS has its input. Financial assistance usually is required, which necessitates getting a project high on the Board priority list. The State OWR can provide useful advice on how best to run the regulatory gauntlet, and where support can be found.

The Orange and Los Angeles Counties Reuse Study will no doubt introduce modifications to this procedure. Identification and planning of projects will be done by this specially assembled group. With a regional viewpoint, projects may well differ from those if strictly local planning had prevailed.

Case for a Regional Reclamation Authority. Following completion of planning efforts, it appears there will be a continuing need for regional planning and management. It appears that these responsibilities could well be given to an agency which would become the regional authority.

The regional authority would be charged with, among other things, seeing that projects are actually initiated at the right time. Changing economic, political and other factors will require timely project starts, but not so far ahead of need as to waste resources and lose political support. Reclamation needs to be integrated into State and Regional water resource planning, and to become a fully utilized portion of the water resource system as remote resources become less available. Surface and groundwater quality impacts must be accounted for, as will land use, growth-inducing, and resultant secondary environmental effects. Pricing of reuse supplies must be competitive with potable supplies, either through reducing some subsidies for present potable supplies or providing comparable subsidies to reuse. For this to be done locally, the regional agency must have taxing authority.

Choosing an agency from among candidates would not be an easy task. Five candidates currently are active at the regional level in the region of concern: (1) the Southern California Association of Governments, (2) an extension of the Orange and Los Angeles Counties Reuse Study organization, (3) the Office of Water Recycling, (4) the Metropolitan Water District, and (5) the regional office of the Department of Water Resources. No single agency at present appears entirely suited to the role. Each has a share of needed expertise and lacks other qualifications. A cooperative effort such as is underway in the current Reuse Study is needed, but with a significantly different goal than the study group has at present. Failure to achieve a coordinated effort most likely will lead to projects that do not provide most efficient use of public money, as well as possible waste of a valuable resource.

CONCLUSIONS AND RECOMMENDATIONS

We have addressed the task of placing information gleaned in our study into proper perspective through the definition of issues: issues which characterize reuse implementation now, which indicate how potentially "better" policy might be derived, issues which suggest more effective implementation. A recommendation derives from each issue statement. Supporting background is very briefly noted here as most has been given earlier in this paper or is presented in the detailed report.

Public Health Policy Regarding Risk

Issue. Current practices in health risk assessment are geared to a zero-risk concept; however, goals set for increasing wastewater reclamation with simultaneous protection of health are not achievable under this concept. Furthermore, logic would dictate that level of risk accepted -- determined by means of water quality standards set -- should be determined from the perspective of derived costs and benefits.

Recommendation. A generalized, systematic and comprehensive methodology needs to be established to analyze data for risk implications and assess potential health risks from individual projects. The methodology should reduce level of uncertainty in data required for standard-setting and assessment of projects, and their utilization. Standards set as a result should take into account economic realities of costs and benefits. They should be reviewed in the context of decreasing marginal returns as well as the more conventional context of increasing protection of health.

Background. This recommendation is based on the belief that systematic, comprehensive analysis would make clearer to the decision-maker, to those directly impacted, and to the general public how and where technical data, value judgements, and assumptions were used in reaching decisions.

Risk Assessment Data

Issue. The data available at present are inadequate to accurately assess the public health risk from wastewater reclamation.

Recommendation. Data are needed in several areas in order to provide a sound basis for the risk assessment which must be made. Areas of particular importance include viruses, heavy metals and organics.

Background. Documented and accurate information is essential to assess health risks of using reclaimed water for any purpose. Without such data no great increase in water reclamation is likely to be realized. Data currently available, while extensive, are not sufficient for application with a high level of confidence.

Standard Setting Responsibility and Practice

Issue. Standard setting responsibility presently merits some clarification and improvement as to procedures, involvement, and implementation. Furthermore, the present narrow framework for developing and implementing criteria is inadequate to provide for all potential uses.

Recommendation. An agency such as the Department of Health Services (DOHS) should have clearly designated responsiblity for definition and measurement of health risks from various reuse alternatives. An agency such as the Water Resources Control Board should have ultimate responsibility for establishing water quality criteria, with provision for modification of requirements as different situations arise. In part this situation exists today. The DOHS is mandated to establish statewide reclamation criteria for each reuse option, and the regional Water Quality Control Boards prescribe discharge requirements based on these criteria. However, legislative intent appears not to have been fully executed: criteria have not been set; there is uncertainty as to division of responsibility in practice. A clear definition of agency roles within a decision-making framework should be established and become effective. DOH's health risk assessment role, and the WRCB's central responsibility to set all water quality standards based on the degree of acceptable risk and economic cost and benefit, can be the initial step.

Background. Objectives are to develop a closer and better defined relationship between health research activities and decision-making, and to obtain a better risk assessment by incorporating a broader range of factors important to the costs and benefits of water reclamation. The conservative approach to risk assessment currently in use is not comprehensive, flexible and accountable to the extent necessary.

Wastewater: A Pollutant or a Resource?

Issue. At present reclamation is treated as more closely related to water quality and associated sanitary engineering practices than to water supply. If reclamation is to provide alternative sources of water, then reclamation must be planned and implemented with a supply orientation.

Recommendation. Planning and implementation of reclamation as part of supply would be best performed by an agency oriented toward supply. The orientation of the Office of Water Recycling (OWR) appears directed more toward implementation of facilities capable of reclaiming wastewater than integration of water, once reclaimed, into supply. Quality concerns should be retained by the Water Resources Control Board.

Background. For reclamation to become an integral part of the state's water supply system it must be planned into existing fresh water management programs. Presently, planning appears oriented either towards water supply or water quality, depending on the agency involved. The

Water Resources Control Board (WRCB) plays a crucial role in dispensing funds. The Office of Water Recycling (OWR) as a project facilitator recommends projects for funding and attempts to mitigate barriers to implementation. Thus it presently has a lead role. The Department of Water Resources is an appropriate home for reclamation planning and integration of reclamation into the fresh water system. The WRCB and OWR are still needed; quality will remain important, but must be balanced with supply perspective.

Need for Integrated Planning and Management

Issue. Present state water resource management is not fully coordinated. Integrated planning and management is State policy; statements to this effect exist in the Water Code. However, supply and quality interests are administered to a substantial degree independently of one another. Incorporation of reclamation into the state's water resource system requires a complete coordination, with integrated planning and management for all water resources.

Recommendation. State policy in the future should address the desirability of total managed conjunctive use in the state (groundwater, surface water, reclaimed water and conservation). Policies should not foster the present dual water orientations perceived at the State level. If statewide integration seems impossible, a regional agency may achieve a more immediate though less extensive conjunctive use program.

Background. The concept of total managed conjunctive use provides an approach for insuring best allocation and most efficient use of the state's water. It will require cooperation of the Water Resources Control Board and Department of Water Resources, which must develop a single value orientation towards water resource planning.

Equitable Cost Comparisons Between Fresh and Reclaimed Water

Issue. Financial comparisons between fresh and reclaimed water projects should provide an accurate, clear and fair picture of both benefits and costs. Agency regulations which constitute cost computation guidelines -- federal, state and local -- should reflect this goal.

Recommendation. A proper delineation of all costs is essential for the comparison of freshwater and reclaimed wastewater projects.

Integration of External Costs Into Economic Analyses

Issue. There are environmental and social costs both in removing freshwater at its origin and in the usage of reclaimed water; external costs should be integrated into the project assessment.

Recommendation. Research programs should be initiated to quantify environmental and social costs of freshwater and reclaimed water projects. Emphasis should be placed on appropriate methodologies and on the collection of data essential to dealing with the costs and benefits of both the use and non-use of a water resource.

Financing of Wastewater Reuse Projects

Issue. Freshwater purveyors develop systems with unused capacity to take advantage of long term needs and economies-of-scale. Large, long-term investments and contracts for specific quality water characterize most delivery provisions. Existing freshwater projects have been financed largely by property taxes through general obligation bonds. Wastewater projects are expected to be financed through Clean Water Grants. Previously incurred fixed costs and debt service of freshwater purveyors may be significantly impacted by reclamation projects.

Recommendation. Avenues of financing that in fact subsidize different types of supply should be examined to ensure a proper balance of public financial burden and to promote an integrated water resource management system. A system of transfer payments should provide just compensation for fixed costs resulting from past good-faith planning by freshwater purveyors. A reallocation of quantity commitments of specific quality water in different geographic regions could be arranged by means of a water bank. Thus cost of increasing the overall efficiency of the state water system through reclaimed water augmentation would not be borne solely by the regional supply agency of the reclamation area. The Department of Water Resources should obtain authority to reallocate funds usually established for added reservoirs for use to finance reclamation projects and supplement freshwater supply.

The Pricing of Fresh and Reclaimed Water

Issue. It appears that there should be a price differential between fresh and reclaimed water since reclaimed water is in general of lower quality, and additional costs often would be incurred for its use.

Recommendation. The price differential between fresh and reclaimed water should be set such that use of reclaimed water is encouraged and more efficient use of freshwater is indirectly promoted. The use of price modifiers such as *ad valorem* taxes should be de-emphasized, however, in order for price to more closely reflect actual costs incurred.

Rights to Sell Reclaimed Water

Issue. It is unclear at present whether a seller of reclaimed water can sell and convey water to a point outside the seller's borders when the plan to sell occurs after original treatment facility construction.

Recommendation. A seller should be given power to sell and convey reclaimed water to a point ouside his borders even though the use plan follows project completion. An owner without explicit authority to sell reclaimed water also should be given that authority.

Background. While state law permits the sale and distribution of reclaimed water from sewage plants, recent action by the State Water Resources Control Board shows that under certain circumstances such a sale may be challenged.

Local Versus Regional, State Reuse Planning

Issue. Water reuse projects tend to be planned and designed by the agency which will operate the system. Such projects tend to be designed to best serve the interests of the operating agency and may or may not be in the best interests of the larger region or the state.

Recommendation. Reclaimed water project planning needs to be done by an agency whose goal is to see that available reclaimed water is put to best use in an efficient manner. This agency should work closely with the various local agencies, but should not itself have parochial interests.

Background. The case studies show that for at least a few areas demand exceeds supply and there is a degree of competition for reclaimed water. Even though each has the best interests of his service area in mind, allocation to a particular purveyor may not yield maximum benefits.

Encouraging Water Reuse

Issue. The State Water Resources Control Board (WRCB) does not have authority to issue cease and desist orders when its regulations interpreting reasonable beneficial use are violated. The courts consider these as mere policy statements. Further, while greater reuse appears to be a goal, the Department of Water Resources does not have authority to review water supply contracts. This may limit reuse opportunities.

Recommendation. The WRCB must have the authority to issue cease and desist orders when its regulations interpreting reasonable beneficial use are violated. Judicial review must be limited to overruling the Board only where such action was arbitrary and capricious. Furthermore, a state-level agency should have authority to review supply contracts in the interest of encouraging increased use of reclaimed water.

Increased Wastewater Reuse: What Are the Choices?

As stated in the introduction, this study has been performed with the assumption that the benefits to be derived from increased wastewater reuse generally outweigh the costs which would accompany them. The facts brought forward do not dispute this assumption. However, three basic areas arise from this work as definite areas of concern. If wastewater reclamation and reuse is to be substantially increased, barriers within these three areas must be overcome.

Health undoubtedly represents a major concern. Much is known about the constituents within water and their possible impact on health. But when it comes to making a reuse decision, a very great deal also is not known. Ideally, specific elements of potential risk would be scientifically accounted for, based on very good (though not perfect or complete) evidence. An assessment methodology would then combine such data to picture, in a clear, statistically significant manner, the likely consequences of a course of action. However, to use such evidence requires that we set standards for specific projects. These project standards may entail greater energy consumption, as one example, and added, secondary risk. Thus, by its very nature, there is a stage at which a balancing of wide-ranging alternatives is performed -- an inherently political process, not presently capable of the thoroughly impartial assessment we can perform in "hard" science areas.

A second major concern is economics. Water is very valuable. But in reality, water does not cost us very much. Thus price probably is not high enough to induce more than reasonable attempts at conservation. No doubt uses of reclaimed water will be constrained for some time to applications fortunately situated so costs can be low. However, a very reasonable course would be to deliberately seek out, and attempt to site in favorable locations, those industrial, water-intensive activities capable of supply by means of reuse; a parallel to what in the energy field is termed "cogeneration". A key factor is the large financial commitment to freshwater projects laid out years ago. To avoid fiscal anarchy, an efficient, equitable system to transfer funds and pay sunk costs must be devised and agreed to.

One further broadly-ranging issue area has emerged from our work as important. This is the need for integrated water resource planning -- planning for conjunctive use of surface water supplies, groundwater, wastewater by means of reuse, and also appropriate water conservation.

It is clear that while calls for wisely managed, conjunctive use of resources have long been heard, a new kind of thinking may have to infuse numerous agencies before many barriers are overcome. Reuse development is a task requiring a hybrid approach -- a combination of concern for technical facts and how these are best mobilized and used in a modern society. We will need the best skills ranging from engineering to microbiology, plus all analytical talent available, just to provide a factual basis from which a relatively small supply agency can make a "best" decision. We also will need institutional cooperation on a scale not characteristic of the past.

Perhaps this last is representative of the most serious institutional barrier to greater wastewater reuse that must be overcome -- the natural tendency to continue without change a basic approach which has worked well in the past. Change appears difficult even when the evidence is clear that adverse impacts from the established approach are growing, and at the same time an innovative alternative is showing greater acceptability. If so, the benefits from even well conceived augmented reuse will come slowly, following substantial efforts.

ACKNOWLEDGEMENTS

The research leading to the report summarized in this paper was conducted under Grant No. 14-34-0001-7814 from the Office of Water Research and Technology, U.S. Department of the Interior; Drs. Laura M. Lake and Richard L. Perrine, Principal Investigators. We wish to express our thanks to Mr. Theodore Roefs and Mr. Robert Medancy of OWRT for continued guidance and cooperation.

The underlying research was performed by a team of UCLA graduate students and faculty: students John I. Baum, David W. Chan, Janet Fahey, Linda L. Fargo, Keith E. Kawaoka, Donald A. Lewis, Joan P. Licari, Mingin Philip Lo, Paul J. Mankiewicz, Sidney F. Paige, Pankaj Parekh, Suzanne L. Phinney, Robert Scofield, Alan J. Silver, Paul E. Smokler, Wayne L. Stone, Carolyn M. Hunsaker, and Jay D. White; and as faculty Dr. Ernest A. Engelbert, Dr. John A. Dracup and Dr. Malcolm S. Gordon together with the Principal Investigators.

Substantial assistance was provided by the project Advisory Committee, both during active project work and by useful, critical review of the draft project report. Organizations represented were the State Water Resources Control Board, the Regional Water Quality Control Board (Region 4), the State Department of Health (Los Angeles office), Office of Water Recycling, Department of Water Resources (Southern District), Los Angeles City Department of Water and Power, the Sierra Club, the League of Women Voters, Aquacare (formerly Watercare), California Association of Water Purveyors, and the Southern California Association of Governments. Individuals also providing counsel are too numerous to include. Their most valuable efforts are sincerely appreciated.

REFERENCES

1. Policy and Action Plan for Water Reclamation in California. Water Resources Control Board. Sacramento, CA. (1977).
2. Executive Order B-36-77. State of California Executive Department. Sacramento, CA. (Oct. 13, 1977).

3. Institutional Barriers to Wastewater Reuse. Final Report, Environmental Science and Engineering, University of California, Los Angeles. Grant No. 14-34-0001-7814, Office of Water Research and Technology, Department of the Interior. (April, 1979).
4. BRYSON, JOHN E. Chairman, State Water Resources Control Board. Testimony before California Senate Committee on Natural Resources and Wildlife, The Extent to Which Reclaimed Water and Groundwater Could be Managed and Enhanced to Meet Future Water Needs. Sacramento, CA. (May 31, 1977).
5. The California Water Plan, Outlook in 1974. Bulletin No. 160-74, Department of Water Resources. Sacramento, CA. (Nov. 1974).
6. Inventory of Waste Water Production and Waste Water Reclamation in California, 1973. Bulletin No. 68-73, Department of Water Resources. Sacramento, CA. (April 1975).
7. ________________. Phase I Report and Work Plan, Orange and Los Angeles Counties Water Reuse Study. Los Angeles, CA (Oct. 17, 1978).
8. The Porter-Cologne Water Quality Control Act. Water Resources Control Board. Sacramento, CA. (July, 1976).
9. Wastewater Reclamation Criteria. Title 22, Division 4, Sections 60301-60357, California Administrative Code.
10. LEE, CLIFFORD T., Legal Aspects of Water Conservation in California. Governor's Commission to Review California Water Rights Law, Staff Paper No. 3. (August, 1977). p. 36.
11. PIERSON, WAYNE. Personal communication from staff of the Office of Water Recycling, Water Resources Control Board. (Aug. 24, 1978).
12. Section 4950 et. seq., California Health and Safety Code. See MOSKOVITZ, Quality Control and Reuse of Water in California. 45 Cal. L. R. (1957). p. 586.
13. Civil No. 49608, Cal. Super. Ct. Placer County. (Sept. 30, 1977).
14. 13 Cal. 2d 343,353.
15. C.P.U.C. Section 1503.
16. P. L. 93-523, as amended by P. L. 95-190.
17. Cal. Gov. Code, Section 11420, et. seq.
18. 23 Cal. Adm. Code, Section 783.
19. 23 Cal. Adm. Code, Section 761(a).
20. LEE, CLIFFORD T., Legal Aspects of Water Conservation in California. op. cit. p. 30.
21. Ibid., p. 32.
22. Institutional Barriers To Wastewater Reuse. op cit. Chap. 6.
23. Reclaimed Water Distribution System, Walnut Valley District. Project Report. Brown and Caldwell, Inc. (March, 1978).
24. CSDOC Wastewater Management Program Environmental Impact Statement, Vol. 2. County Sanitation District of Orange County and U.S. Environmental Protection Agency. (March, 1977 Draft).
25. Final Environmental Impact Report on the Long Range Energy Program for the California State Water Project. Department of Water Resources. Sacramento, CA. (1977).
26. New Water or Reclaimed Water. Office of Water Recycling, Water Resources Control Board. Sacramento, CA. (Oct. 1977).
27. Reclaimed Water Distribution System, Walnut Valley District. Project Report. Brown and Caldwell, Inc. (May, 1978). p. 8-18.
28. P. L. 92-500, 92nd Congress, S. 2770. Office of Public Affairs, U.S. EPA. Washington, D. C. (Oct. 18, 1972).
29. P. L. 95-217. Digest of Public General Bills and Resolutions, 95th Congress, 2nd Session, 1st Issue. Washington, D. C. (1978).
30. Annual Report, The Metropolitan Water District of Southern California. (1977). Appendix, p. 154.
31. Institutional Barriers to Wastewater Reuse. op. cit. Appendix B.

32. PERRINE, R. L., et. al., Wastewater Reclamation and Reuse. Proc. Water Policy Conference. Water Resources Center, University of California and the Kellogg Foundation. (1979).

33. BRUVOLD, W. H. Public Attitudes Towards Reuse of Reclaimed Wastewater. Water Resources Center Contribution No. 137. School of Public Health, University of California, Berkeley. (1972).

34. Wastewater Reclamation: Socioeconomics, Technology, and Public Acceptance. Ralph Stone and Co., Inc., for Office of Water Resources Research, U.S. Department of the Interior. Washington, D. C. (May, 1974).

35. DAVOS, C. A. A priority-Tradeoff-Scanning Approach to Evaluation of Environmental Management. Jour. Environ. Management 5:3:259-273. (1977).

36. DAVOS, C. A. Environmental Policy Evaluation Within a Social Context: The Necessity of Synthesis. Jour. Environ. Systems 7:3:201-214. (1977-78).

37. Annual Survey Report on Ground Water Replenishment, Central and West Basin Water Replenishment District. Bookman-Edmonston Engr., Inc. (1978). p. 29.

38. Final EIS/EIR, Joint Outfall System Facilities Plan, Vol. I. Los Angeles County Sanitation Districts. (1977). p. I-71 to I-78.

39. Plan of Study, Southern Area Water Reclamation Project, Pomona Valley Municipal Water District. Pomona Valley Municipal Water District and Engineering-Science, Inc. (May 18, 1978).

40. Plan of Study for Reclamation Water Conveyance Facility. Engineering-Science, Inc. (May, 1976).

41. OWEN, LANGDOM W. Proposed Pilot Injection of Treated Sewage Effluent to Water Bearing Aquifer by the OCWD. Report from the Secretary-Manager, OCWD, to the California State Board of Public Health. (March 26, 1965).

42. Water Factory 21. Orange County Water District. (1977).

43. Annual Report. Orange County Water District. (April, 1977).

44. WHITTENBERG, W. R. and ARGO, D. G. Imported Water and its Energy Impacts in Orange County. Fourth Annual Conference, National Water Supply Improvement Association. (July 11-15, 1976).

TABLE 1

Data Summary for Surrogate Savings-Demand Curve in Southern California

Type of Reuse	Potential Demand (AF/yr)	Required Savings ($/AF)	Required Savings (¢/m^3)
Los Angeles County			
Groundwater Replenishment	140,000	7[c]	0.6
Landscape Irrigation			
Schools	280	28[b]	2.3
Others	41,644	38[b]	3.1
Industrial	110,320	62[b]	5.0
Sea Water Intrusion Barrier	39,200	306[c]	24.8
Oil Field Repressurization	8,960	?	?
TOTAL	340,480		
	($4.20 \times 10^8 m^3/yr$)		
Orange County			
Groundwater Replenishment	112,000	7[a]	0.6
Landscape Irrigation	39,088-44,688	38[b]	3.1
Recreation	2,240	38[b]	3.1
Industrial	12,544	62[b]	5.0
Agricultural	12,992	62[b,d]	5.0
Sea Water Intrusion Barrier	33,600-39,200	306[c]	24.8
TOTAL	212,464-223,664		
	($2.62-2.76 \times 10^8 m^3/yr$)		

[a] LACSD charge for treatment and delivery costs

[b] Reference 23

[c] Cost of AWT including electrodialysis, Reference 24

[d] Required savings reflects need for long-term commitment and restriction of crop type

TABLE 2

Cost Components for Fresh and Reclaimed Water

Fresh Water	Reclaimed Water
Capital Costs (Fixed Costs)	
Conservation Facilities • Reservoirs and Power Plants	Usually None, though Possible
Major Transportation Facilities • Peripheral Canal • Aqueduct and Pump Stations • Storage Facilities and Conjunctive Use • Recovery Power Plant	Usually None, though Possible
Distribution System • Major Lines (Water Wholesalers) • Local Lines (Water Retailers)	Distribution System • Trunk Lines • Local Lines • Small Storage or Conjunctive Use Facilities
Water Treatment Facilities	Additional Treatment Facilities Internal Conversion to Reclaimed Water Use
Operation and Maintenance Costs (Variable Costs)	
Aqueduct Delivery Costs • Pumping Costs • Power Recovery Credit	
Water Treatment Costs	Additional Treatment Costs
Local Delivery Costs (Minimal Pumping)	Local Delivery Costs (Considerable Pumping)
	Quality Reliability Costs (To Users)

TABLE 3

Projected Costs and Firm Yields From New Conservation Facilities

Project	Critical Period Yield AF/Yr	Cost[1] $/AF (July 1977)
Los Banos Reservoir	240,000	150
Los Vaqueros	160,000	250
Los Vaqueros-Los Banos (Tandem Opr.)	390,000	350
Enlarge Lake Berryessa	790,000	109
Glen Reservoir	990,000	107
Tuscan Buttes	640,000	252
Colusa Reservoir	460,000	172
Cottonwood Project (Dutch Gulch and Tehama Reservoirs)		90
Enlarge Shasta	990,000	114
Valley Tributary Storage (10 Reservoirs)	700,000	89
Dos Rios Reservoir	930,000	118
English Ridge Reservoir	210,000	224
New Melones Reservoir[2,3]	245,000	26
Auburn[2]	318,000	115
Marysville Reservoir[3,4]	155,000	27
ALL PROJECT AVERAGE	--	158 (12.8¢/m^3)

---

[1] Delivered to the Delta

[2] Federal Projects Authorized and Under Construction

[3] The cost per acre-foot of these two supplies is abnormally low due to power generation. The average cost without these projects is about $158/AF (12.8¢/m^3).

[4] Federal Project Authorized.

Source: Reference 26.

TABLE 4

Projected Marginal Costs of New Freshwater Delivery to the South Coast Basin Area of California
(Costs in $/Acre-Foot/Year)

Capital Costs (1975 dollars)	Averaged Costs, All New Supply Projects	Marginal Project (Los Vaqueros)
Conservation Facility	158	250
Major Transportation Facilities		
Peripheral Canal	4	4
Aqueduct and Pump Station[1]	(sunk cost)	(sunk cost)
Distribution System	(sunk cost)	(sunk cost)
SUBTOTAL	162	254
	(13.1¢/m^3)	(20.6¢/m^3)

Aqueduct Costs (1985 dollars)	DWR Pumping Cost Estimate[2]	Energy Commission Low Estimate[3]	Energy Commission High Estimate[3]
Aqueduct Net Pumping Costs	45	112	196
Water Treatment Cost[4]	25	25	25
Local Distribution O&M Cost[5]	121	121	121
SUBTOTAL	191	258	342
	(15.5¢/m^3)	(20.9¢/m^3)	(27.7¢/m^3)

---

[1] A 1965 DWR study of the capital costs to convey an additional 885,000 AF annually from the delta to the Tehachapi Tunnels indicates a 1975 cost level expenditure of about $665/AF, equivalent to $47/AF (3.8¢/$m^3$) at 7 percent and 50-year life.

[2] Estimate from DWR Bulletin 132-77, Castaic Reach (1985).

[3] California State Energy Commission, Biennial Report (1978), Table II-39; estimates for Southern California Edison (assuming 3400 KWH/AF) are a low at 3.3¢/KWH, and a high at 5.77¢/KWH.

[4] MWD, 1978 Pricing Study, Report No. 931 (March 1978).

[5] Based on Reference 27.

TABLE 5

Unit Process Treatment Costs for Wastewater

Process	Cost ($/Acre-Foot)	Cost (¢/m^3)
Disinfected Secondary Treatment[1]	70	5.7
Advanced Treatment		
Chemical Coagulation (Phosphorus Removal)	30	2.4
Filtration	25	2.0
Nitrogen Removal	35	2.8
Carbon Adsorption	45	3.6
Demineralization	130	10.5
Spreading, Storing and Withdrawing from Underground Reservoir (Conjunctive Use)	65	5.3

---

[1] Federal law requires secondary treatment. Disinfection is also normally required in discharge permit for compliance with California Department of Health regulations. Therefore, these costs are not included as additional costs for reclamation.

Source: Reference 26.

TABLE 6

Projected Costs of Required Treatment for Various Reclaimed Water Uses

Reclaimed Wastewater Use	Treatment Required	Additional Cost[1] ($/AF)	Additional Cost[1] (c/m^3)
I. Agricultural Irrigation			
(a) Spray irrigation of food crops	Secondary treatment and disinfection, chemical coagulation and filtration	55	4.5
(b) Irrigation of fodder, fiber and seed crops	Secondary treatment and disinfection	0	0
II. Landscape irrigation (golf courses, freeway landscapes, etc.)	Secondary treatment and disinfection	0	0
III. Industrial use (cooling water process water, etc.	Secondary treatment and disinfection, coagulation and filtration	55	4.5
	If demineralization is required	185	15.0
IV. Groundwater recharge	Disinfection and carbon adsorption (proposed)	45	3.6
V. Recreational Impoundment			
(a) Unrestricted	Disinfection, coagulation and filtration	55	4.5
(b) Restricted	Disinfection	0	0
(c) Landscape	Disinfection	0	0
VI. Deep subsurface injection	Disinfection, coagulation filtration, demineralization (projected)	185	15.0

[1] The unit process treatment costs for wastewater in the previous table were employed.

Source: Reference 23.

TABLE 7

Summary of Projected Costs for Reclaimed Water
Walnut Valley Water District

(Limited Reuse, Alternative C)

Capital Costs	$/AF
● Distribution System	203
● Distribution Lines Storage Reservoir Conjunctive Use Facilities	
● Internal User Conversion	$2,000 to 2,500*
● Additional Treatment Facilities	Not Required
O & M Costs	
● Operations--Pumping	39
● Maintenance	24
● Quality Reliaiblity Costs (User)	Not Considered
TOTAL (Excluding User Cost)	266 (21.6¢/m^3)

---

* Cost per user, minimum; not cost in $/AF

Source: Reference 27.

TABLE 8

Average Annual Sources of Groundwater Replenishment For Central and West Coast Basins
(Adapted from Reference 38)

Supplier	Source	Amount of Replenishment			Percent Of Total
		(AFY)	(AFY)	($10^6 m^3/yr$)	
Metropolitan Water District	imported water		130,000	160	46
1. surface spreading		75,000			
2. salt-water barriers (deep well injection)		55,000			
Los Angeles County Sanitation District (Whittier Narrows and San Jose Creek WRP's)	imported water		25,000	31	9
(Natural)	rainfall and stream flows		125,000	154	45
1. sub-surface		25,000			
2. rising water		20,000			
3. storm water		15,000			
4. rainfall		65,000			

TABLE 9

Source of Water in the Study Area

Sources of Water Purveyed Within the Pomona Valley Municipal Water District Service Area

Source	FY 1976/77		FY 1977/78	
	Amount (AF)	Percent	Amount (AF)	Percent
MWD	31,433.4	39.1	31,964.7	40.9
Ground Water	45,873.4	57.0	41,409.4	53.0
Surface Water	3,185.4	3.96	3,513.9	4.50
Reclaimed Water	625.2	0.78	1,238.9	1.58
	80,502.2	100	78,126.9	100
	$(99x10^6m^3)$		$(96x10^6m^3)$	

Sources of Water Purveyed by the City of Pomona For FY 1977/78

Source	Amount (AF)	Percent
MWD	1,608.9	6.34
Ground Water	18,846.6	74.3
Surface Water	3,514.9	13.8
Reclaimed Water	1,398.0	5.51
TOTAL	25,368.4	100
	$(31x10^6m^3)$	

Sources of Water Purveyed by the Walnut Valley Water District for FY 1976

Source	Amount (AF)	Percent
MWD	8,820.5	100
	$(11x10^6m^3)$	

TABLE 10

Potential Users of Reclaimed Water from the WVWD Line

Potential User	Estimated Usage, 1976-1980	
	acre-ft/yr	$10^3m^3/yr$
City of Industry*	1,000	1,233
County of Los Angeles Golf Course	400	493
County of Los Angeles Otterbein Park	100	123
Los Angeles County Sanitation Districts Spadra Sanitary Landfill**	50	62
Pomona National Golf Course	400	493
Day and Night Company	100	123
Ajax Hardware	150	185
Queen of Heaven Cemetery	100	123
Freeway Landscaping	100	123

* The City of Industry has plans to develop two 18 hole golf courses and one 9 hole golf course and a park area.

** The plans of the County Sanitation Districts of Los Angeles County include development of a golf course upon completion of the sanitary landfill.

Source: Reference 40.

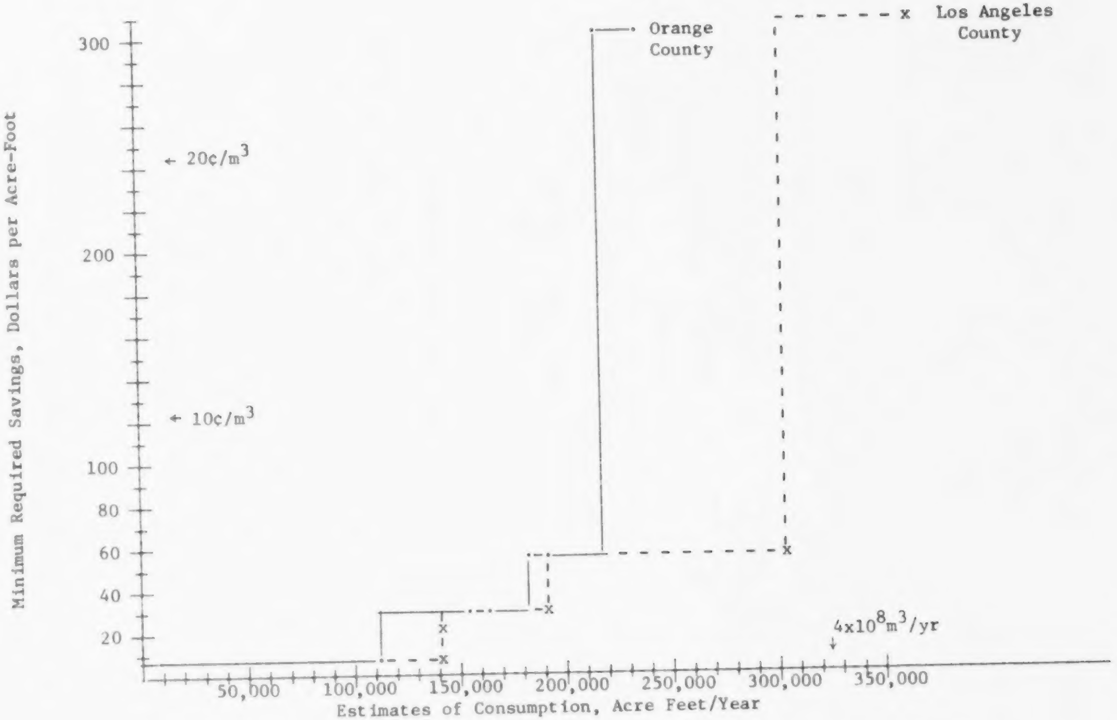


FIGURE 1

Surrogate Savings-Demand Curve for Reclaimed Wastewater in Southern California

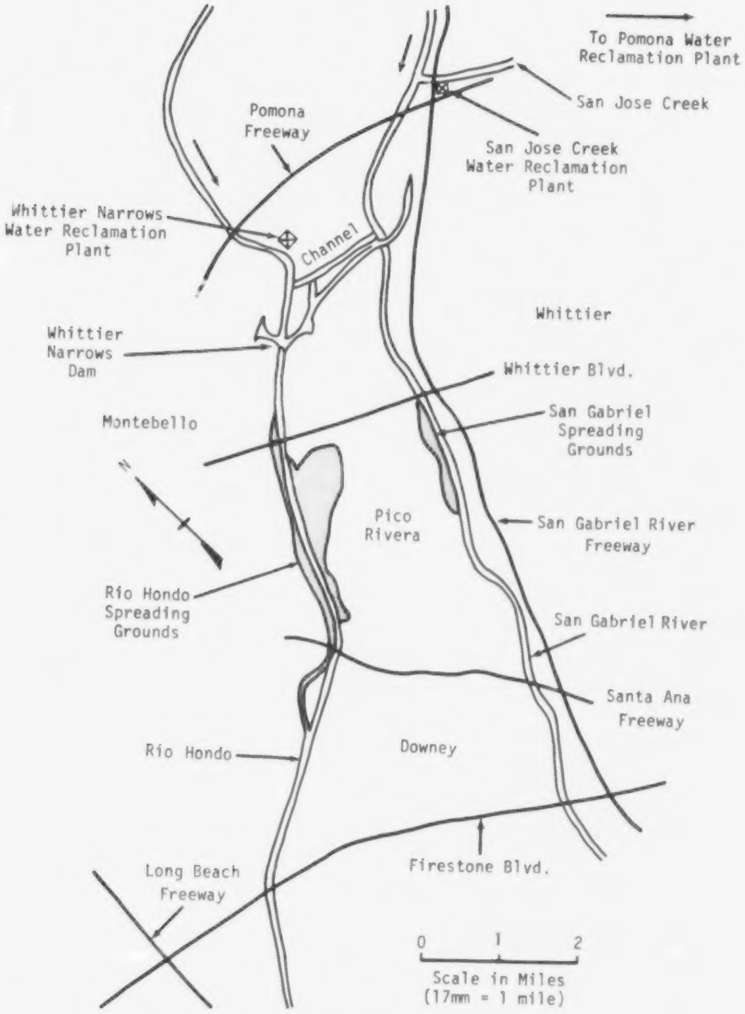


FIGURE 2

San Jose Creek Water Reclamation Plant and Spreading Grounds

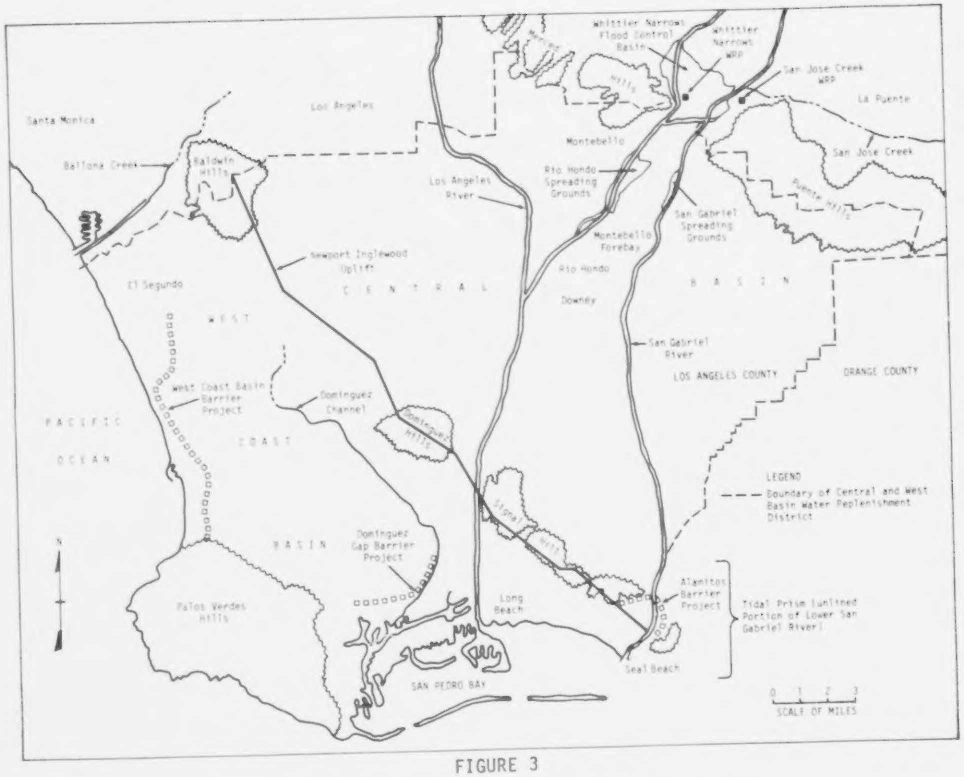


FIGURE 3

Central and West Basin Replenishment District

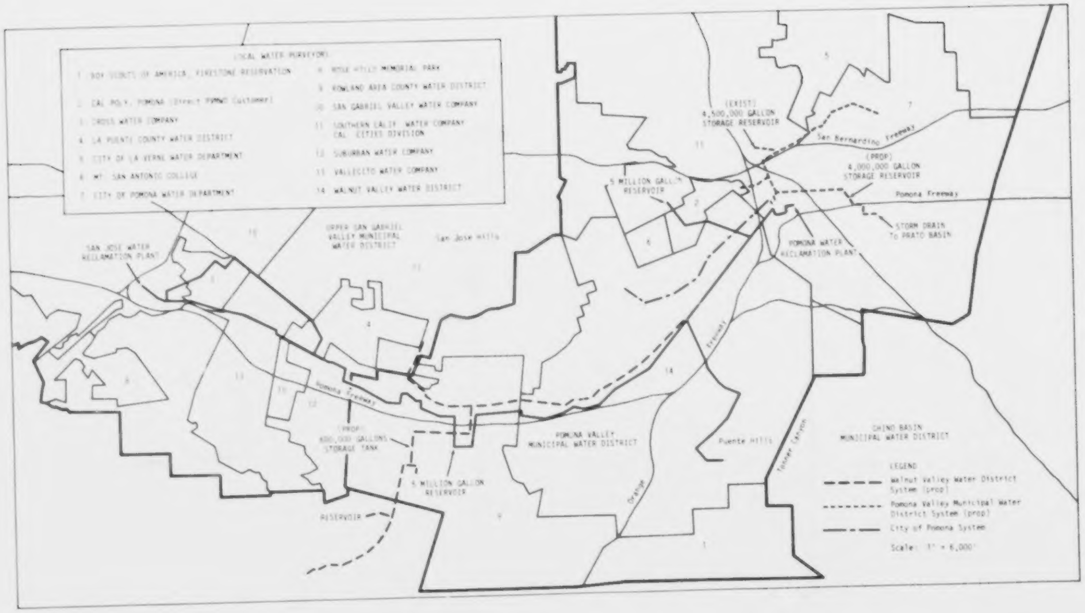

LOCAL WATER PURVEYORS
1 BOY SCOUTS OF AMERICA, FIRESTONE RESERVATION
2 CAL POLY, POMONA (Direct PVMWD Customer)
3 CROSS WATER COMPANY
4 LA PUENTE COUNTY WATER DISTRICT
5 CITY OF LA VERNE WATER DEPARTMENT
6 MT. SAN ANTONIO COLLEGE
7 CITY OF POMONA WATER DEPARTMENT
8 ROSE HILLS MEMORIAL PARK
9 ROWLAND AREA COUNTY WATER DISTRICT
10 SAN GABRIEL VALLEY WATER COMPANY
11 SOUTHERN CALIF. WATER COMPANY CAL. CITIES DIVISION
12 SUBURBAN WATER COMPANY
13 VALLECITO WATER COMPANY
14 WALNUT VALLEY WATER DISTRICT
SAN JOSE WATER RECLAMATION PLANT
UPPER SAN GABRIEL VALLEY MUNICIPAL WATER DISTRICT
San Jose Hills
5 MILLION GALLON RESERVOIR
(EXIST) 4,500,000 GALLON STORAGE RESERVOIR
San Bernardino Freeway
(PROP) 4,000,000 GALLON STORAGE RESERVOIR
Pomona Freeway
STORM DRAIN To PRATO BASIN
POMONA WATER RECLAMATION PLANT
Freeway
Pomona Freeway
(PROP) 600,000 GALLONS STORAGE TANK
5 MILLION GALLON RESERVOIR
RESERVOIR
POMONA VALLEY MUNICIPAL WATER DISTRICT
Orange
Puente Hills
Tonner Canyon
CHINO BASIN MUNICIPAL WATER DISTRICT
LEGEND
Walnut Valley Water District System (prop)
Pomona Valley Municipal Water District System (prop)
City of Pomona System
Scale: 1" = 6,000'
1
2
3
4
5
6
7
8
9
10
11
12
13
14

FIGURE 4

Walnut Valley-Pomona Water Reclamation and Service Area

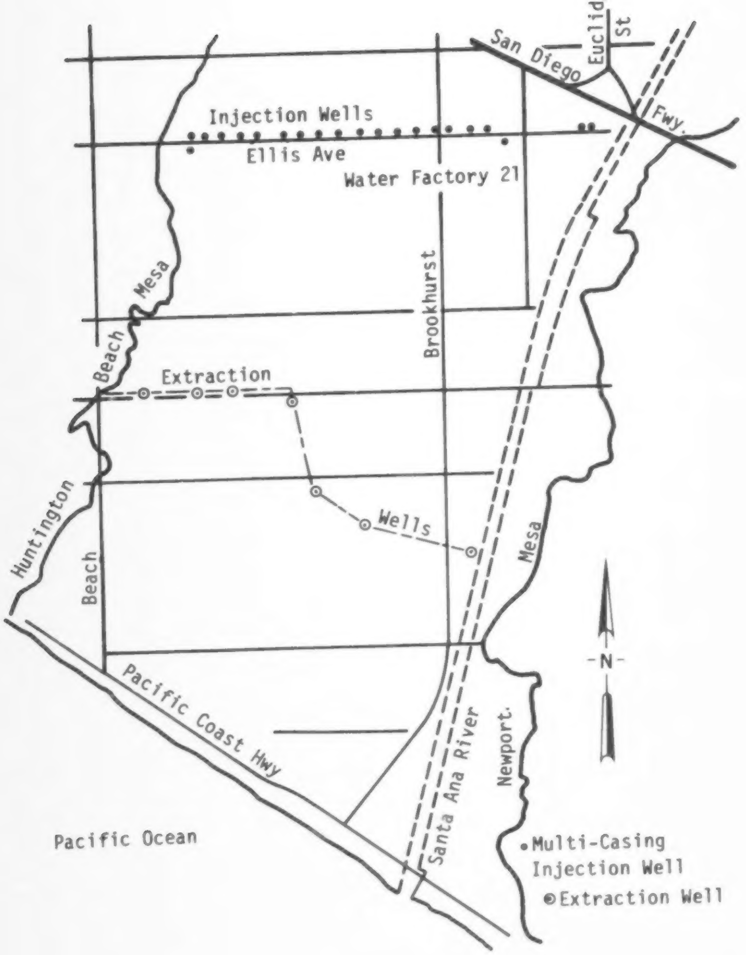


FIGURE 5

Orange County Water District Coastal Barrier Project

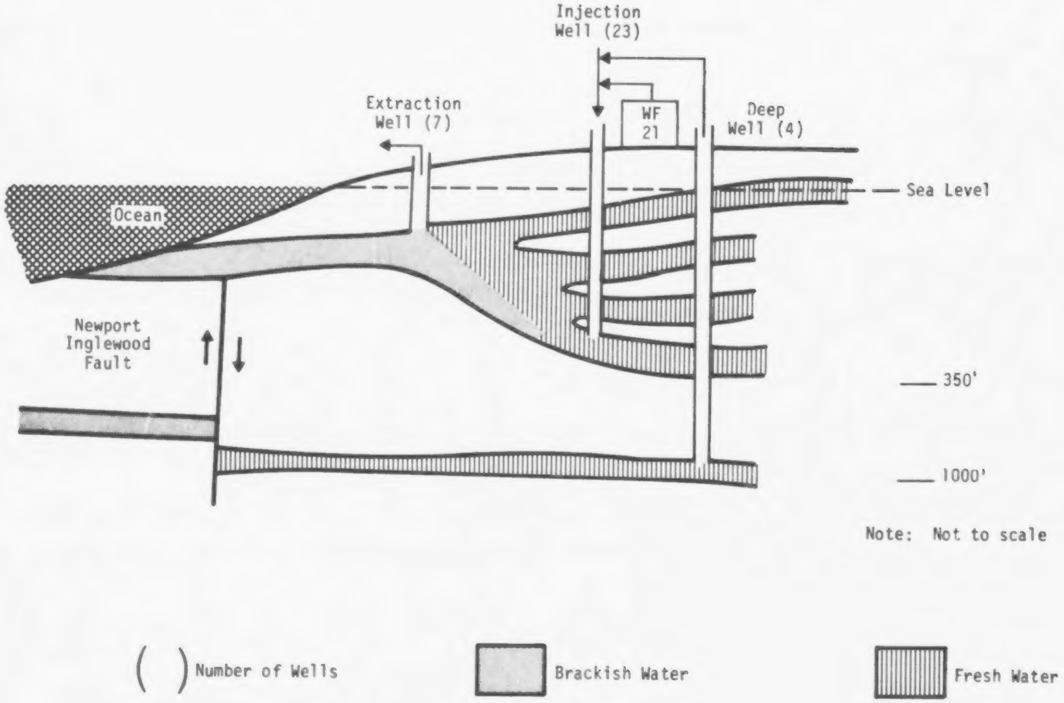


FIGURE 6

Schematic Cross-Section of Water Factory 21 Barrier Project

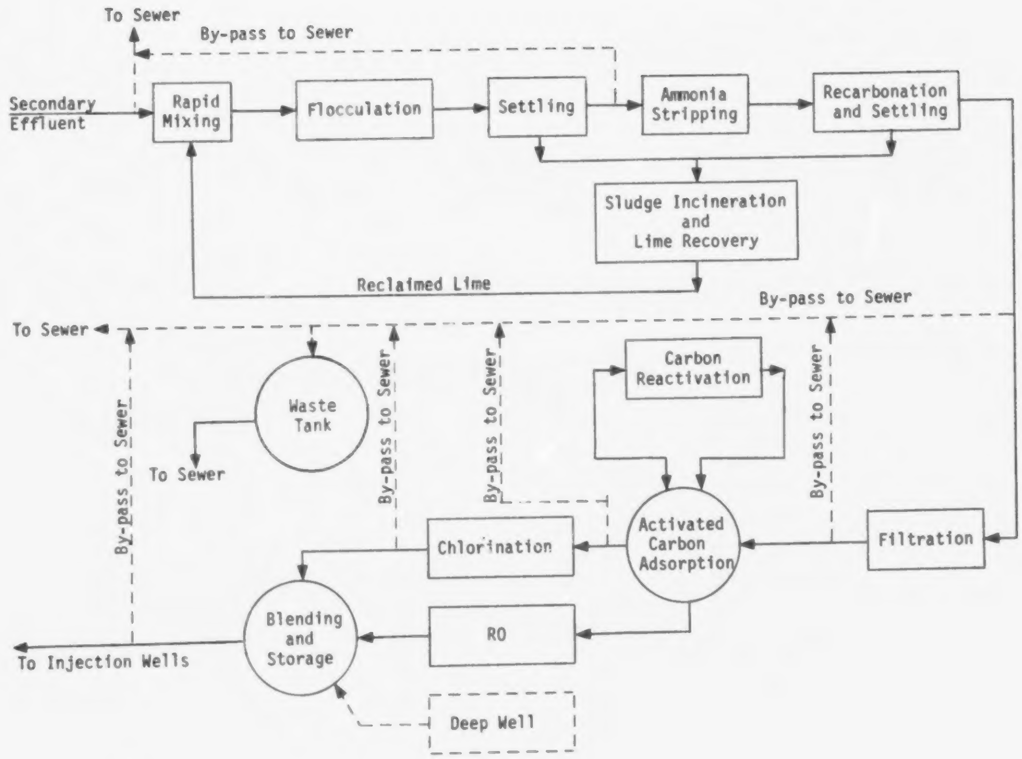


FIGURE 7
Water Factory 21 Reclamation Process Flow Diagram

ECONOMICS OF WATER REUSE IN METROPOLITAN WATER SUPPLY AND WASTEWATER MANAGEMENT

Sashi Kumar Mozumder
Graduate Student
College of Engineering
Division of Hydraulic and
Sanitary Engineering
620 Davis Hall
University of California
Berkeley, California 94720

Charles R. Scherer
Assistant Professor
College of Engineering
Division of Hydraulic and
Sanitary Engineering
University of California
Berkeley, California 94720

In many western metropolitan areas water supply development has been characterized by a series of increments to water import capacity. In each case it was necessary to go farther (or deeper) from the demand point than the previous source augmentation, and this has resulted in an increase in real cost of delivered water at the demand point. Part of this increased cost was due to added pumping energy required to deliver the water through a greater distance. Furthermore, due to future anticipated increases in the cost of energy, the next cycle of sypply augmentation will be achieved at even greater costs, not only because the transport distance is farther but also because of rapidly rising cost of energy. Also, since water resources in the West are finite, and since popular opinion supports preservation of free running rivers (e.g. Northern California Rivers), new conventional "sources" are becoming limited. These are some of the pressures that have moved western water resources planners towards more thoughful planning of conjunctive use of surface and groundwater and trends that require a comprehensive "systemwide" approach to metropolitan water-wastewater management more ingenius in "use" and "reuse."

Recently, municipal wastewater treatment requirements (secondary and greater) have been mandated by law (PL 92-500); the cost of meeting them may be viewed --- from an economic accounting perspective --- in a similar way to costs that are considered sunk. That is, they represent resources already committed and therefore irrelevant for future allocation decisions. However, there remain any costs of further upgrading effluent quality to qualify various effluents as sources of reuse. These latter resources are not yet committed, so their social value is pertinent to decisions on metropolitan reclamation and reuse policy. As this event took place, it has been noted by regulatory agencies (e.g. California State Water Resources Control Board, 1977), and used as a basis for advocating water reclamation and reuse wherever "economically feasible." With this mandate by powerful agencies such as SWRCB (and U.S. Congress in PL 92-500) it becomes necessary to postulate operational criteria of "economic feasibility" and demonstrate methods of determining the extent to which, and circumstances under which, reuse is desirable according to this criterion. This is a major purpose of this research paper.

Currently, the rising cost of energy has been recognized as a major factor in the future of water-wastewater management. Energy cost is pertinent in water and wastewater treatment and conveyance and also in storage by groundwater recharge because the water must be recovered from storage using energy. Hence, the rising cost of energy will influence, in the long-run, the selection of reclamation and reuse alternatives. This role of increasing energy cost in metropolitan water-wastewater planning can best be evaluated using the dynamic, price-sensitive demand methodology demonstrated by this study. A dynamic planning methodology is necessary to correctly characterize and incorporate the changing value of energy over time into water treatment and wastewater reclamation facility staging. Although, some argue that municipal water-wastewater service demand is not especially responsive to historically observed price changes, it may be more sensitive to future water-wastewater prices which could increase dramatically as energy costs increase. Therefore, to accurately assess the role of energy costs in influencing reclamation, reuse and "conservation,"[1] a price-sensitive version of the existing minimum cost reuse studies is presented.

A PLANNING MODEL ON ECONOMICS OF WATER REUSE IN METROPOLITAN WATER SUPPLY AND WASTEWATER MANAGEMENT

Given a set of potential local and import alternatives for future water supplies and also wastewater reclamation and reuse alternatives within the framework of regional plan, the modeling problem becomes that of developing a selection methodology for choosing efficient projects to meet potable and non-potable demands and their corresponding prices within a cost-benefit framework. A large scale, mixed-integer programming model was developed to handle the large quantities of relevant informations to obtain a multi-faceted development plan with a prior knowledge of the following factors.

1. Costs of increasing capacities of alternative projects (water and wastewater).
2. All operating and maintenance costs for water supply and wastewater services (all costs divided into energy and non-energy components).
3. Capacity limitations of each project.
4. Prices of water-wastewater services and their price elasticities at specified consumption levels in different time periods.
5. Controls on pollution of receiving water body/bodies.
6. Local reservoir release policies.
7. Limitations on imports.
8. Annual/and cumulative budget constraints.

Layout of the Model:

In the generalized version of the model (Figure 2) we consider the system received surface and subsurface supplies from various import sources (HH, BA, and SF). The system has its own local supplies -- surface (SY) and subsurface (PK). The water is either conserved in local ground-water basin (Q) by the recharging process (RG) or stored in surface reservoir(s) system. Water released from these storages goes to the consumer sectors (K) directly or via water treatment plant (m). Once used, wastewater is captured by sewers and conveyed to primary and secondary wastewater treatment plants (ES) from where it has the option to go to one of these advanced wastewater facilities DS (for discharging to the receiving water body), IN (for injection to groundwater Q), SP (spreading in the recharge basin RH) and RU (direct reuse in non-potable sectors).

---

[1] Conservation here means reduction of use or demand.

Applications and Illustration of Solutions

In this section an example problem is set up to illustrate the use of the model. The region chosen for this purpose is the northern portion of the Santa Clara Valley (Fig. 2). Santa Clara Valley Water District is the wholesaler of water in this fast developing metropolitan area consisting of several communities. It imports water from two different sources in addition to its own surface and groundwater supplies. The options available for future water development to meet the growing demands of the region are a) development of a new import source, b) increase local reservoir yields, and c) reuse wastewater directly from advanced wastewater treatment plants or indirectly by recharging it to the ground water aquifers.

The water demands in this area are for residential, commercial, industrial and irrigational purposes. The first three sectors form the potable water demands while the last one forms non-potable demands. Geographically this region is bounded to the north by San Francisco Bay, to the west and southwest by the Santa Cruz mountain range, to the south by a narrow valley gap at Morgan Hill and to the east by the Diablo range. The total drainage area of the basin is about 500 square miles and the aquifer -- underlain in the basin occupies an area of 216 square miles (18 miles x 12 miles). Currently, no systematic reclaimed water reuse (direct or indirect) is practiced in the region. Only recently a pilot project has been set up in the regional water pollution control plant located at Palo Alto to inject and recover advanced treated wastewater to achieve dual purposes -- i) create salt water barrier to protect underground supply and ii) to provide water for landscaping and irrigation, and for limited industrial uses after passing it through the aquifer medium. Currently, wastewater from this region is treated up to the secondary level before discharging into the San Francisco Bay. According to a California Water Resources Control Board study, wastewater discharged into the bay has to be advance treated for biological nitrification. Also recently, the Santa Clara Valley Water District is looking into the possibility of reusing the reclaimed water from the regional water pollution control plants which is otherwise released to the bay. The costs involved for the proposed development are mainly pipelines, pumping and storage facilities and some preliminary polishing of the secondary treated wastewater. However, the costs of groundwater recharge by spreading and by injection of reclaimed water are quite expensive. Therefore future selection of such alternatives need proper economic evaluation and consideration of their long-term effects. Using the northern portion of Santa Clara Valley as the example, it is the intention of the authors to describe in a general manner the uses of the model for evaluating the important economic decisions related to the water-wastewater planning of the region.

In order to illustrate the type of information obtained by the solution of the model, excerpts of actual solutions are presented in Tables 2, 3, and 4. To obtain some meaningful and comparable results, a "controlled model" solution was first obtained with the data input made available from the Santa Clara Valley Water District. Some of the basic data input of the model are presented in Table 1. In the "controlled model" options for wastewater reuse are limited to irrigation and landscaping only. This is in accordance with the view of the Santa Clara Valley Water District's current management plan. Indirect use of reclaimed water reuse by spreading and by injection are ruled out to avoid any long-term health hazard. Also, in this controlled solution scheme, the Valley's projected (demands)/requirements are met at the price set by the water and wastewater utilities. Therefore, in this controlled solution no decision had to be made on quantities consumed and their associated prices. The purpose of this "controlled model" solution is to compare the rest of the solutions with this as a detum and to illustrate the economic efficiency of other schemes in terms of water use and reuse.

With the controlled model solution as detum, next the decision on wastewater reuse is relaxed allowing the model to choose the most economic reuse alternatives. However, the decision on consumptions and their associated prices are kept fixed as in the controlled solution. Table 2 illustrates the difference in project selections between these two schemes. Indirect reuse by spreading is indicated to be more economically efficient that direct reuse. This, however, does not account for possible long-term damage on ground water quality due to long-term recharging of reclaimed water. Tables 3 and 4 illustrate the difference in a controlled solution scheme against the optimal solution scheme. In the optimal scheme the model is allowed to choose all the potential reuse alternatives and also to determine consumption levels and their associated prices. The optimal scheme indicates that it is economical to reduce consumption by increasing prices in the residential and commercial sector and some price and consumption readjustment in other consumer sectors, without any reuse.

Summary

The interrelated nature of physical, technical and economical parameters in a water-wastewater system makes modeling complicated but it helps planners and operators of water-wastewater systems as a tool to answer specific questions of interest under "what if" situations. This paper has dealt with a general metropolitan water-wastewater system model oriented towards the evaluation of efficient reuse alternatives within the regional wastewater management policies. The economic framework presented coordinates the two basic phases of metropolitan water-wastewater systems (water supply, treatment and distribution at wholesaler's level; and wastewater treatment and management), in order to derive economic assessment of wastewater reclamation and reuse.

Table 1. Some Basic Data Input.

1. Time horizon = 24 years.
2. Construction periods = 4 (every 6 years).
3. Demand periods = 8 (every 3 years).
4. Water-wastewater prices and elasticities (whole-saler's level)
   a. Potable Sector
      (i) residential and commercial

	Summer	Winter
Price	$195/Ac. Ft.	$195/Ac. Ft.
Elasticity	- 0.38	- 0.26

      (ii) industrial

	Summer	Winter
Price	$240/Ac. Ft.	$240/Ac. Ft.
Elasticity	- 0.6	- 0.8

   b. Non-potable, Agricultural and Non-agricultural Irrigation Sector

	Summer	Winter
Price -	$39/Ac. Ft.	$39/Ac. Ft.
Elasticity	- 0.9	- 2.7

5. Discount rate = 2.0% (uninflated).
6. Grant from federal and state government for wastewater facilities - 87½%.
7. Return flow co-efficients
   i) from residential and commercial users
      Summer = .55
      Winter = .85
   ii) from industrial user
      Summer = .85
      Winter = .85
8. Irrigation efficiency of non-potable consumers = 70%.
9. Growth rate of tax revenue = 2% annually.
10. Energy (real) cost increase rate = 2% annually.
11. Energy price 1.5 cents/kilowatt hour.

Table 2

Efficient Timing and Sizing of Water-wastewater Projects[a]
with and without "Controlled Model" Relaxed for Reuse Alternatives.

Facility	Controlled Model				Relaxed Controlled Model			
	Building Period				Building Period			
	1	2	3	4	1	2	3	4
1. local pumping capacity	65,800	19,110	15,600	11,360	65,800	19,110	15,000	11,300
2. wastewater capacity								
a. primary & secondary	-	10,960	9,670	7,030	-	10,900	9,670	7,030
b. advanced treatment for spreading	-	-	-	-	49,470	-	-	-
c. advanced treatment for injection	-	-	-	-	-	-	-	-
d. advanced treatment for direct reuse in non-potable sector	65,800	-	-	-	-	-	-	-
e. advanced treatment for discharge to bay	100,000	-	-	-	100,000	-	-	-
3. new pipeline capacity to import	-	-	17,460	-	-	-	-	-

---

[a] Capacity build in each building period, acre feet/season. To convert to million gallon per day divide by 560.00.

Table 3. Pricing and Consumption Obtained from "Optimal Model" Solution.

Time Period	Potable Sector Price[a]				Non-potable Sector Price[a]	
	Residential & Commercial Sector		Industrial Sector		Irrigation Sector	
	Summer	Winter	Summer	Winter	Summer	Winter
1	279.24 [195.00]	7.49 [195.00]	222.36 [240.00]	128.05 [240.00]	39.00 [39.00]	28.22 [39.00]
2	296.42 [195.00]	43.10 [195.00]	273.51 [240.00]	150.17 [240.00]	39.00 [39.00]	30.35 [39.00]
3	311.84 [195.00]	166.89 [195.00]	303.70 [240.00]	165.00 [240.00]	39.00 [39.00]	31.77 [39.00]
4	323.28 [195.00]	360.53 [195.00]	333.95 [240.00]	165.00 [240.00]	39.00 [39.00]	31.77 [39.00]
5	323.29 [195.00]	369.31 [195.00]	391.75 [240.00]	165.00 [240.00]	39.00 [39.00]	31.77 [39.00]
6	323.28 [195.00]	382.50 [195.00]	429.48 [240.00]	165.00 [240.00]	39.00 [39.00]	31.77 [39.00]
7	323.28 [195.00]	382.50 [195.00]	187.24 [240.00]	165.00 [240.00]	39.00 [39.00]	31.77 [39.00]
8	323.28 [195.00]	361.94 [195.00]	228.97 [240.00]	165.00 [240.00]	39.00 [39.00]	31.77 [39.00]
	Consumptions [in Acre Feet]					
1	136,098 [162,830]	89,775 [71,822]	29,583 [28,735]	15,842 [12,676]	29,060 [29,060]	7,144 [4,763]
2	138,146 [172,180]	99,412 [82,569]	28,258 [30,384]	18,235 [14,588]	26,799 [26,779]	6,679 [4,452]
3	140,195 [181,528]	97,019 [93,515]	26,932 [32,034]	20,627 [16,502]	24,498 [24,498]	6,211 [4,141]
4	142,243 [189,658]	76,140 [97,704]	25,607 [33,469	21,551 [17,241	22,851 [22,851]	5,874 [3,916]
5	148,341 [197,789]	78,210 [101,893]	21,661 [34,903]	22,475 [17,980]	21,204 [21,204]	5,536 [3,691]
6	152,612 [203,493]	78,619 [104,826]	18,897 [35,908]	23,122 [18,498]	20,824 [20,824]	5,460 [3,640]
7	156,882 [209,177]	80,818 [107,758]	41,791 [36,913]	23,770 [19,016]	20,443 [20,443]	5,382 [3,588]
8	160,152 [213,537]	85,518 [110,005]	38,721 [37,683]	24,265 [19,412]	20,158 [20,158]	5,323 [3,549]

Values within brackets are the "controlled model" solution results.

[a] Prices include water and wastewater services in dollars/Acre foot.

Table 4

Efficient Timing and Sizing of Water-wastewater Projects[a] from Optimal Solution Scheme.

Facility	Controlled Model[b]				Optimal Model[c]			
	Building Period				Building Period			
	1	2	3	4	1	2	3	4
1. local surface storage	-	-	-	-	-	-	-	-
2. local pumping capacity	65,800	19,110	15,600	11,360	29,640	-	2,990	26,900
3. local recharge capacity	-	-	-	-	-	-	-	-
4. water treatment capacity	-	-	-	-	-	-	-	-
5. wastewater capacity								
a. primary & secondary	-	10,960	9,670	7,030	-	-	-	-
b. advanced treatment for discharge to bay	100,000	-	-	-	100,000	-	-	21,600
c. advanced treatment for spreading	-	-	-	-	-	-	-	-
d. advanced treatment for injection	-	-	-	-	-	-	-	-
e. advanced treatment for direct reuse in non-potable sector	65,600	-	-	-	-	-	-	-
6. new pipeline capacity to import	-	-	17,460	-	-	-	-	-

---

[a] Capacity built in each building period, acre feet/season. To convert to million gallon per day divide by 560.00.

[b] The net present value of the costs associated with the scheme for the 24-year time horizon was estimated approximately 910 million dollars (1980 $).

[c] The net present value of the costs associated with the scheme for the 24-year time horizon was estimated approximately 700 million dollars (1980 $).

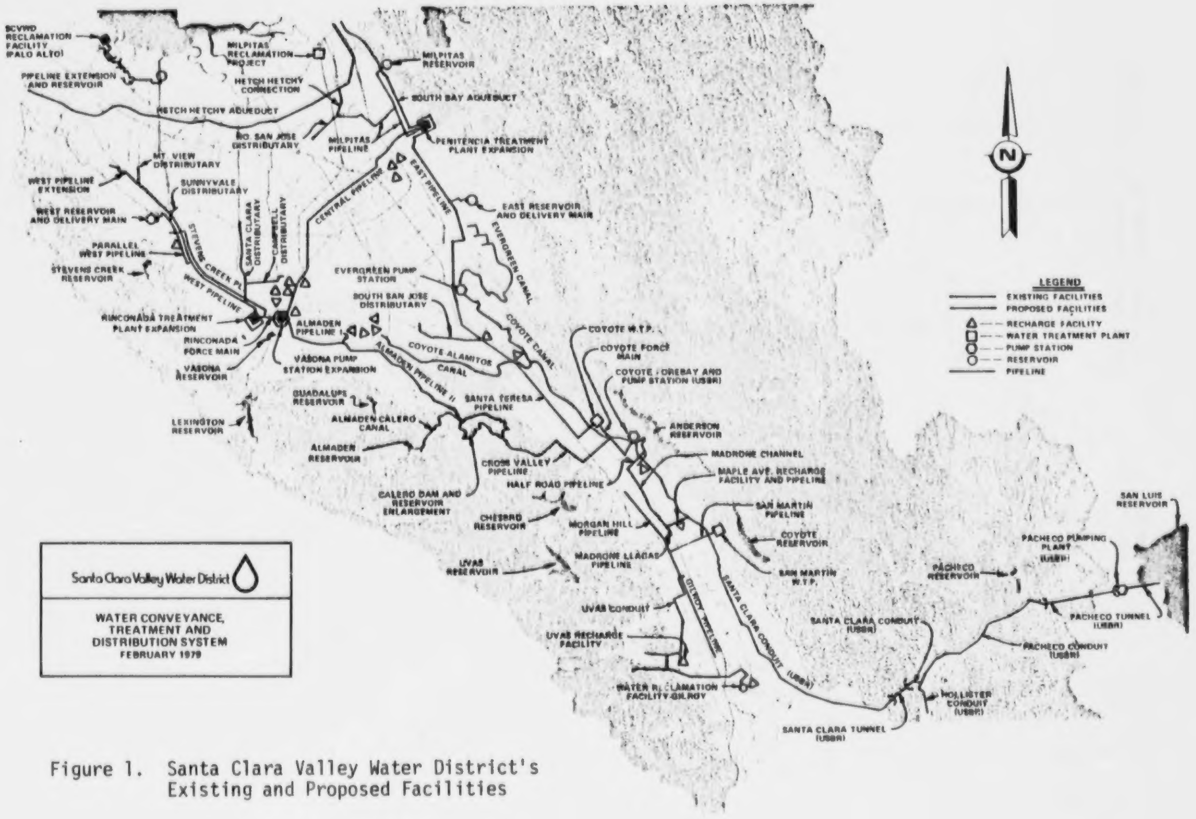


Figure 1. Santa Clara Valley Water District's Existing and Proposed Facilities

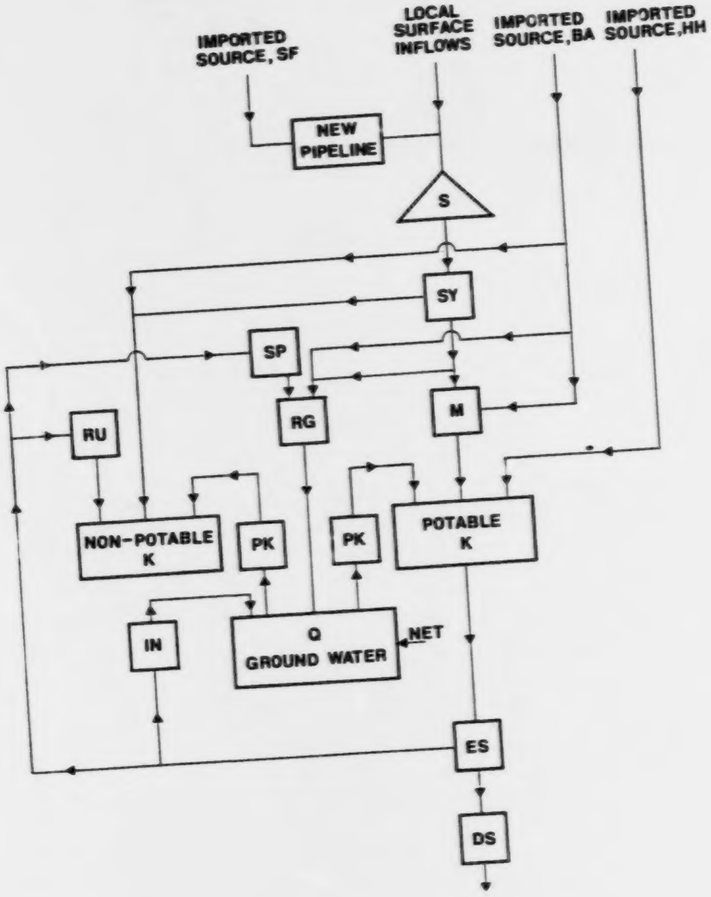


Figure 2

Schematic Layout of the Generalized Model

Bibliography

1. Bishop, A.B. & R. Narayanan, "Seasonal and Stochastic Factors in Water Planning," *Journal of the Hydraulic Division*, vol. 103, No. HT.10, Oct. 1977.

2. Bishop, A.B.; B.C. Jensen & R. Narayanan, "Economic Assessment of an Activity Analysis Model for Water Supply Planning," *Water Resources Research*, 11(6), 7[illegible]1-88; Dec. 1975.

3. Bishop, A.B.; D.W. Hendrics & S.A. Milligan, "Water Reuse System Analysis," *Water Resources Research*, 7(3), 542-53, June 1971.

4. Burges, S.J. & Maknoon, R., *A Systematic Examination of Issues in Conjunctive Use of Ground and Surface Waters*, C.W. Harris Hydraulics Lab., University of Washington, Seattle, Washington; October 1975.

5. California Department of Water Resources, *Waste Water Reclamation*: State of the Art, Bulletin 189, 1973.

6. California State Water Resources Control Board, *Policy and Action for Water Reclamation in California*; January 1977.

7. California State Water Resources Board, Bulletin No. 7, *Santa Clara Valley Investigation*, June 1955.

8. California State Water Resources Control Board, *Guidelines for the Estimation of Total Energy Requirements of Municipal Wastewater Treatment Alternatives*, August, 1

9. Dworkin, D.M., "Water Reuse: A Flexible and Efficient Management Alternative for Municipal Water Supply," *Water Resources Research*, 11(5), 607-15, October 1975.

10. Dracup, J.A., *The Optimum Use of a Ground-Water and Surface Water System: A Parametric Linear Programming Approach*, Hydraulic Laboratory, University of California, Berkeley, Water Resources Center Contribution No. 107, July 1966.

11. Gysi, Manshall, *The Long Run Effects of Water Pricing Policies*, Cornell University Water Resources and Marine Sciences Center, Ithaca, New York, March 1971.

12. Hirshleifer, Jack; James C. DeHaven, & Jerome W. Milliman *Water Supply-Economics, Technology and Policy*, The University of Chicago Press, 1960.

13. Hughes, Trevor C, *A Mixed Integer Programming Approach to Planning Multiple Water Sources for Municipal Water Supply*, Utah Water Research Laboratory/College of Engineering, Logan, Utah; March 1972.

14. Howe, Charles W., & Linaweaver, F.P., "The Impact of Price on Residential Water Demand and Its Relation to System Design and Price Structure," Water Resources Research, 3(1), 13-32, 1st Quarter 1967.

15. Lynne, Gary D.; William G. Luppold & Clyde Kiker, "Water Price Responsiveness of Commercial Establishments," Water Resources Bulletin, American Water Works Association, Vol. 14, No. 3, June 1978.

16. Mulvihill, M.E. & J.A. Dracup, "Optimal Timing and Sizing of a Conjunctive Urban Water Supply and Wastewater System with Nonlinear Programming," Water Resources Research, 10(2), April 1974.

17. Okun, Daniel, "Principles for Water Quality Management," Journal of the Environmental Engineering Division, ASCE, 103(EE6), 1039-56, December 1977.

18. Schelhorse, L.D., et al., The Market Structure of Southern California Water Industry, Copley International Corporation, La Jolla, California, June 1974.

19. Santa Clara Vally Water District, Master Plan - Expansion of In-county Water Distribution System, December 1975.

20. Santa Clara County Flood Control and Water District, Draft Water Resource Master Plan Interim Summary Report of Water Demands, Cost Functional, Facility Capacities, Distribution Systems for Existing Facilities, San Felipe, Percolation Alternatives, July 1973.

21. U.S. Environmental Protection Agency, Office of Water Program Operations, Washington D.C., 20 460, EPA-430/9-75, A Guide to the Selection of Cost Effective Wastewater Treatment Systems, July 1975.

22. U.S. Department of the Interior, Office of Water Resources Research, The Importance of Economic Factors in Urban Surface Water Supply Systems, June 1972.

23. Walker, W.R.; G.V. Skogerboe, & R.C. Ward, "Water Supply Treatment, Distribution and Reuse in Arid Urban Areas," Water Resources Bulletin, 10 (6), December 1974.

THE COSTS FOR SPRAY IRRIGATION LAND DISPOSAL OF MUNICIPAL WASTEWATER IN CALIFORNIA

Dale R. Hurd
Ph.D. Candidate
Department of Economics
University of California, Riverside, 92521

William W. Wood, Jr.
Economist
Cooperative Extension
University of California, Riverside, 92521

Victor B. Youngner
Professor of Agronomy
Department of Plant Sciences
University of California, Riverside, 92521

Gary B. Benoit
Staff Research Associate
Cooperative Extension
University of California, Riverside, 92521

THE SETTING

In the past the primary focus of the wastewater district has been to dispose of the flow for which it is responsible subject to the various cultural, economic, environmental, institutional, and legal constraints placed upon it. Currently we are witnessing a change in this "disposal" philosophy due to the synergistic influence of several factors, primarily the increasing population and burgeoning technological sophistication. In many regions one now finds that the assimilative capacity of the environment has been unable to meet the increased demands placed upon it by population pressures and unacceptable levels of environmental degradation have resulted. Concomitantly the questionable economic viability of supplying ever increasing amounts of "fresh" water has led to serious doubts about the wisdom of our current water system.

The short-run approach of dealing with these problems has led to the imposition of additional institutional and legal constraints in an attempt to relax the binding economic and environmental ones. In response to these changing constraints new forms of water supply and water disposal agencies are evolving to provide a viable long-run solution to the problem at hand. The metamorphis emphasizes the advantages to be realized from agency intergration and the avoidance of artificial boundaries whereby water changes from "fresh" to "waste." The general recognition of water quality as a continum transversing several dimensions has given a new impetus to using water which is acceptable in the restricted critical dimensions for the limited use to which it is to be put. In this regard spray irrigation using wastewater is uniquely promising because it not only produces benefits from the harvesting of the crop cover but is capable of producing a high quality water "harvest" as well.

THE PROBLEM

This research attempted to quantify the specific monetary and energy costs associated with land application renovation of wastewater, and to a lesser extent consider the long-run implications.

Monetary Costs

If the various non-economic constraints are to be met by a given regimen of wastewater disposal, which procedures result in the least cost method of operation? Van Note (1) lists 123 combinations of unit processes for wastewater treatment systems ranging from primary sedimentation to ion exchanges and associates with each an expected water quality level. However, in addition to the great diversity of treatment systems from which to choose Van Note states that, "there are wide cost variations caused by factors unique to any given project, e.g., site conditions, local variations in material and labor costs and different wastewater characteristics." It is to be expected that research intended to have broad applicability for diverse geographical regions should inform the reader of the nature of the cost estimates, such as this caveat from Middleton (2), "Costs given are examples and do not apply to any specific local situation." But when one enters the realm of wastewater treatment by land application one encounters another problem of similar magnitude. Pound (3) noted in 1975 that, "For the most part. . .the costs were predominantly built up from typical preliminary designs since very few actual construction cost data are available for existing land application systems. It is hoped that actual costs can be used to a greater degree in future revisions of this report as more data become available." In a more recent work by Young (4) he notes the same problem and states his approach, "Simulation analysis was selected as the mode of analysis for three reasons: (1) simulation analysis permits examination of land application treatment under a wide variety of scenarios, (2) the direct impact of individual parameters can be observed, and (3) since only a limited number of land application systems are being operated for advanced wastewater treatment, insufficient data are available for statistical analysis."

Circumstances novel to California have acted to mitigate these data restrictions: i) because over 80 percent of water use in California is for irrigation there was an early and strong interest in this form of renovation, ii) the depletion of groundwater aquifers and the subsequent intrusion of saltwater in certain coastal regions stimulated interest in various recharge methods, iii) the establishment of the State of California Office of Water Recycling, and iv) the extensive and costly canal system which traverses much of the state. For these reasons, and the innovative spirit of the local communities which sponsored land application renovation, there exists within the state a small but growing data base upon which to analyze the current cost situation. The authors believe that by restricting the geographical boundaries of the study area, statistically meaningful results may be obtained from the data which are currently available. Consequently the analysis conducted below is based upon actual costs incurred, but no claim is made for the general applicability of the results beyond the study area.

Energy Costs

Another characteristic of California which is assuming increasing significance has given rise to the second thrust of this study; energy requirements. Throughout the state fuel costs for utilities companies are consistently above the national average, while current expectations are that energy prices in the United States will outpace most other price increases in their steady march upward. Although the United States Government implemented the Federal Mandatory Fuel Allocation Program which grants wastewater handling facilities 100 percent of their requirements during fuel shortages, there is a great opportunity for energy conservation by designing and operating energy efficient wastewater treatment systems. The State of California Energy Task Force (5) recognized the serious nature of the problem when it issued this warning: "Because wastewater collection and treatment is vital to the health and general well-being of Californians and is also an energy dependent process, the present energy shortage has potentially serious implications regarding the ability of California to pursue the goal of clean waters." As energy costs assume an increasingly important role in operations costs they will determine the cost effectiveness of treatment systems to a greater extent than ever before.

The authors envision future research utilizing previously published estimates of construction and operation costs for advanced wastewater treatment systems, and the results reported in this research, analyzing the trade-off between the higher construction costs and the lower operation costs associated with land application renovation. This analysis would proceed using appropriate amortization periods and discount rates to determine the impact energy cost escalation has upon the overall cost effectiveness of land application renovation in the long-run. In order to account for the differential rates of price escalation in the construction and energy sectors it would be desirable to develop a simulation model similar in nature to the "CLAW" (Cost of Land Application of Wastewater) model reported upon by Young (4) which could incorporate into the analysis alternative hypotheses concerning these volatile parameters.

PREVIOUS RESEARCH

Much of the previous research into the costs of advanced wastewater treatment has neglected land application renovation and instead has attempted to quantify the costs associated with mechanical and chemical renovation. As a comparison with the results reported here for land application renovation some of the more recent cost estimates for advanced wastewater treatment are summarized.

Monetary Costs

Middleton's estimates (2) for capital and total operating costs of 10 MGD (million gallon per day) plants which produce the quality of water listed is summarized below in 1974 dollars:

WATER QUALITY	CAPITAL ($/mil gal)	TOTAL OPERATING ($/1000 gal)	($/acre-ft)
Recreational Lakes	9,641,000	.407	133
Industrial Uses	8,237,000	.356	116
Near-Potable Water	13,181,000	.621	202
Near-Potable Water by Reverse Osmosis	11,532,000	.623	203

Alternatively the California Water Resources Control Board's estimates (6) for a 10 MGD wastewater treatment facility is summarized below in dollars based on the EPA construction cost index of 200:

PROCESS	COST/1000 GAL (each process)	(cum.)	COST/ACRE-FEET (each process)	(cum)
Secondary	.20	.20	65	65
Chemical Coagulation (lime)	.09	.29	29	94
Filtration	.07	.36	23	117
Nitrogen Removal	.11	.47	35	152
Carbon Adsorption	.10	.57	33	185
Disinfection	.01	.58	3	188
Demineralization	.40	.98	130	318

A recent EPA publication (1) gives annual combined sludge and liquid process costs for various degrees of quality ranging from 38.9 to 60.5 cents/1000 gallons. These estimates are based on the National Average Wastewater Treatment Plant Cost Index of 177.5 for February 1973, the Wholesale Price Index for Industrial Commodities of 120 for February 1973 and labor costs of $5.00/hour. This estimate is for a 20 MGD wastewater treatment plant design for biochemical oxygen demand, suspended solids, phosphorus and nitrogen removal with effluent polishing.

An engineering analysis (7) for a proposed project to be completed in 1983 yields cost estimates for a 15 MGD wastewater treatment plant utilizing spray irrigation land application for groundwater recharge. This project is to receive secondarily treated water as it's influent and thus will exhibit lower costs than plants which must process raw influent. The secondarily treated influent is to be received and treated by coagulation (aluminum sulfate is the primary coagulant and a polymer is the secondary coagulant) followed by multimedia filtration and then disinfection by chlorine. The estimated cost, based on the Engineering News Record's Construction Cost Index of 4200 is:

The Green Acres Project (15 MGD in 1983)

TOTAL CAPITAL COST	OUTPUT (acre-ft/year)	O & M AND CAPITAL AMORTIZATION	UNIT COST ($/acre-ft)
$27,153,000	7,961	$1,016,000/year	$128

In addition to these cost estimates the study cites a reduction in energy demand due to the project because imported water requires 3 to 5 times as much energy to obtain as does reclaimed water.

Energy Costs

These costs consist primarily of electrical energy with some natural gas being used to power heaters, incinerators, and vehicles. In this study gasoline costs were not included in the energy analysis but were placed in the O & M analysis section. While the major energy expense is for electrical consumption, procresses utilizing natural gas may find energy costs increasing at an exceedingly rapid pace when deregulation of natural gas in the United States occurs. A 1973 report sponsored by the EPA (8) summarizes electrical power requirements for 10 MGD wastewater treatment plants, by type of process:

LEVEL OF TREATMENT	ENERGY CONSUMPTION (KwH/mil gal treated)	$ COST (@$.03/KwH)
Primary	235	7.05
Secondary		
Primary + Trickling Filters	480	14.40
Primary + Activated Sludge	880	26.40
Tertiary		
Secondary + Lime Clarification, Filtration, and Carbon Adsorption	1630	48.90
Secondary + Filtration, and Reverse Osmosis	3000	90.00

Based upon these estimates an increase in the price of electrical energy of just one mill per kilowatt-hour would directly increase the operating costs of a 10 MGD wastewater treatment plant operating at capacity and utilizing secondary treatment plus filtration and reverse osmosis in the amount of $10,950 per year. In contrast a 10 MGD wastewater treatment system utilizing primary and activated sludge treatment with a gravity flow spray irrigation land renovation system would incur additional operating costs of only $3,212 per year. These are direct effects only, one would expect the generally higher operating costs of advanced wastewater treatment plants to increase more than proportionally in response to energy price increases due to the indirect and induced effects which these price increases would precipitate. One must also consider the hardships imposed by employing energy intensive treatment systems in light of the U.S. Federal Mandatory Fuel Allocation Program which gives wastewater handling systems a high priority fuel allocation.

For comparison with the actual energy costs reported below one must also include collection system energy costs. The EPA (9) estimates of collection system energy costs updated by the fourth quarter 1978 fuel and utilities component of the National Consumer Price Index are:

ANNUAL COLLECTION SYSTEM POWER COSTS ($/MGD)

(Minimum)	(Median)	(Average)	(Maximum)
194	1018	3,086	24,903

Adding the collection and treatment system's energy costs one arrives at the total system energy cost expressed here as total annual cost per MGD flow:

ANNUAL ENERGY COST RELATIONSHIPS ($/MGD)

SYSTEM	COST		
	(Min.)	(Avg.)	(Max.)
Primary	2767	5659	29088
Secondary			
Primary+Trickling Filters	5450	8432	31771
Primary+Activated Sludge	9830	12722	36351
Tertiary			
Secondary+Lime Clarifi., Fil., and Carbon Adsor.	18042	20934	44363

Secondary+Filtration, and Reverse Osmosis	33044	35936	59365

METHODOLOGY

The actual costs incurred by a broad spectrum of wastewater districts in California for fiscal year 1977 were analyzed by least squares regression and factor analysis in an attempt to determine the relationship between the observed costs and the underlying influencing factors. In addition special attention was given to the relationship between treatment facility type and the associated level of energy consumption per unit of effluent flow.

Two recent developments lend especial relevance to the results of this project. First, the EPA's mandate that new wastewater treatment systems must investigate the cost effectiveness of land disposal of treated wastewater. Second, the findings of studies conducted in New England which indicate that the failure of engineers to anticipate the great energy-cost escalation and to design energy-conserving wastewater systems has increased operating costs to municipalities by 40 to 90 percent.

Data Sources

The State of California Water Resources Control Board was contacted and helpful assistance was received from both the Division of Water Quality and the Office of Water Recycling. From information contained in the control board's data base 18 new and 3 expansion wastewater treatment systems utilizing land application renovation were identified. These facilities were contacted and general information regarding system operations and specific operation and maintenance cost data were obtained. To obtain construction cost data the consulting engineers engaged for the construction of these facilities were contacted for copies of the Bid Tabulations submitted by contractors. There was no uniform procedure for handling this information by the different engineering firms and in many instances the authors were referred to the municipalities for the actual construction cost data. In a few cases it was necessary to proceed directly to the construction firm contracted for the system erection. Three of the new systems were deemed by the authors to have inadequate data or insufficient operating experience to give meaningful results and were dropped from the study group. The final study group consists of 15 new systems and 3 expansion systems; located as indicated in Figure 1.

Indexing

The construction costs incurred for the 15 new plants and the original construction and expansion costs for the 3 older plants were all stated in terms of the November 1978 Engineering News Record Construction Cost Index for Los Angeles and San Francisco of 3,421 and 3,412 respectively. Systems in the Los Angeles sphere of influence were adjusted with the Los Angeles index while systems in the San Francisco sphere of influence were adjusted with the San Francisco index.

Because the operation and maintenance costs were all from the uniform period of July 1, 1977 to June 30, 1978 no indexing of these costs was attempted. Similarly, energy requirements were gathered for this uniform period in physical units and no indexing was required for this part of the analysis.

The Analysis

Using an IBM 360/50 computer and the SAS Institute's Statistical Analysis System software package a combined cross-section time-series regression was run on the effluent flows and energy consumption by the Fuller and Battese Method (10) to obtain a baseline estimate of energy consumption for the study population as a whole. Next, each system's effluent flow rate and energy consumption rate was analyzed by the method of ordinary least squares regression using Princeton University's Time Series Processor software package. Each system was then compared to the baseline estimate for preliminary categorization by energy usage. Having placed the systems into energy requirements per unit processed groups, the authors proceeded to determine which influences gave rise to the between-group variations in energy requirements.

Using the Statistical Package for the Social Sciences software package on a Prime 400 computer, linear regression, crosstabulation, and factor analysis were implemented. This subdivision of the statistical analysis provides the major portion of the results presented below which define the fundamental characteristics of plant types and make inferences about the underlying influences which determine the actual costs incurred. The results are therefore presented within a framework of unit cost functions relative to capacity, treatment system type, topography, et cetera. The complete list of influences is given in the section presenting the results of this study.

The summary results of the statistical analysis may be observed from tables 4 through 8 at the end of this report. These represent the skeletal frame around which the body of the research is structured.

The Data

The data represents the actual costs for the present configuration of the land disposal treatment systems studied. Construction costs were obtained from the consulting engineers, municipal administrations, and construction firms. Land costs were not included in the construction cost figures. In the cases of system expansion major repairs which extended the capital life of the operating equipment were indexed and included in system construction cost rather than maintenance costs. Operation and Maintenance costs were obtained from the wastewater districts in all cases. Administrative and overhead expenses were included in the operations and maintenance costs.

RESULTS

Because utilities costs represent about 10 percent of the operations and maintenance costs for the systems studied the total energy requirements of the individual systems were converted into British Thermal Units and then into KwH equivalent for easy cost comparison based upon a price of 30 mills per KwH. The results indicate that, on the average, land application renovation compares favorably with other forms of advanced wastewater treatment, however, the study group did display wider variations than the comparison group. Primarily due to the influences of the excess capacity, average operations and maintenance costs showed a disadvantage when compared with the EPA (9) estimates of annual average operations and maintenance cost for a 10 MGD system. The EPA's third quarter 1977 annual cost estimate of $145270 per MGD flow is lower than the author's fourth quarter 1978 annual cost estimate of $220117 per MGD flow.

Construction costs are presented with respect to their influencing factors with a view toward analyzing the trade-off between higher initial investment vs. lower O & M costs. Obviously the choice of discount rate, amoritization period, and volatility of O & M costs will influence the ultimate results significantly.

Energy Costs

The summary energy requirements for the study group are:

Annual Energy Costs Per MGD Flow

	MINIMUM	AVERAGE	MAXIMUM
Energy in Mil BTU	132	2518	18276
$ Cost @ $0.03/KwH	$1160	$22128	$160644

Based upon the estimated regression equations presented in Table 5 one finds that total energy requirements are extremely sensitive to the flow entering the system: an additional 1 MGD inflow requiring an additional 1787 Mil BTU. However, contrary to previously reported estimates, the results here indicate that average energy costs are higher for larger system designs, while the total construction cost is relatively insensitive to the energy intensitivity of the system configuration. If these preliminary findings are borne out the high average costs associated with constructing small scale AWT systems could be overcome by employing economical land application renovation. Some support is lent to this hypothesis by the relationship found between energy consumption and total horsepower associated with the treatment systems because increasing average costs were also found in their relationship. As a whole, however, economies of scale did prevail in the study group in terms of operations and maintenance costs. Energy costs are not presently a large enough component of total O & M costs to out-weigh the other savings gained from increased system capacity. But these savings must be traded off due to the fact that systems operating at below capacity levels have been shown to have significant increases in their average operating costs when compared with costs at capacity levels of operation.

No significant relationship was found between total energy consumption and the system type in the study group in terms of the unit processes employed and the treatment train unit process configuration. The results indicate that the over-riding significant factor in energy consumption is pumping costs. While there does exist a relationship between pumping requirements and the treatment train unit process configuration this influence is small when compared to the impact of the other influencing factors. As indicated below in the section on Operation & Maintenance costs there is a significant positive correlation between the system type and the average O & M costs per MGD, but to attribute this to energy requirements is not supported by the data. One hypothesis consistently advanced by system administrators was that in the wake of the 1973 energy crisis managers have attempted to substitute other inputs for energy. The result has been that the hypothesized relationship between energy requirements and system type does not show up in the analysis of the energy data, but instead has been shifted to total O & M costs which do show a significant relationship.

Operations & Maintenance Costs

Average O & M costs are heavily influenced by the degree of capacity utilization of the system. All of the study group facilities had some degree of excess capacity and the resulting O & M costs are therefore somewhat high. As the reclaimed water is recycled additional flows will be available to be processed through the system utilizing the now excess capacity and lowering the average O & M cost. The concept of a "water flow multiplier" is helpful in noting how this result is forthcoming. As each new unit of water is added to the system a portion of this flow is reclaimed and added to the flow in the next period. A portion of this second period flow is reclaimed and again placed in the flow in the third period. Thus in each succeeding period some fraction of the previous period's reclaimed flow is again reclaimed and adds to the available supply in the following period. For illustrative purposes assume that one-half of the flow in any given period is reclaimed. Then an increase in the flow in the present period of one unit will result in a one-half unit increase in the flow in the next period and a one-fourth unit increase in the flow in the third. Or, 1+1/2+1/4+1/8+1/16+1/32+etc., which is a well known series and in the limit converges to 1/(1-1/2)=2, or more generally as; injections divided by the fraction of leakages. Thus the initial injection of one unit of water into the system will result in a two unit increase in water flow, assuming half of the flow is reclaimed in each cycle.

Operation and maintenance costs were estimated to be $209,357 annually for each MGD flow, with fixed costs of $107,607 annually. For a 10 MGD system this amounts to annual average costs of $220,117 per MGD. The relationships between average O&M costs and system type is displayed in tables 6,7,&8. Wide variations were observed in O & M costs which were only partly attributable to the variations in the energy intensiveness of the systems, other contributing factors are discussed below.

Capital Costs

The relationship found between total construction cost and system capacity was rather weak, with wide variability in the data encountered. Interestingly there was a stronger, but still weak, relationship between the total capital cost and the actual system flows. The summary capital costs are:

Capital Costs

	Fixed Costs	Marginal Costs/MGD	Avg. Costs/MGD
Observed Flows	$5,348,907	$1,348,060	$1,882,950
Capacity Flows	4,562,976	811,796	1,268,093

Cost Variations

Exceedingly wide cost variations were encountered in all the categories analyzed. To account for the factors influencing this variation alpha factor analysis was employed. To determine the extent to which planners could employ this variability to good advantage the analysis is presented in two sections: the collection/processing system and the spray irrigation distribution system.

Collection/Processing

The greatest influence on the cost variations was system capacity. This factor accounts for 45 percent of the cost variations in the study population. In general the larger the system design flow the lower the average capital and O & M cost per MGD flow at capacity utilization levels. But there are severe penalties associated with under utilization of the design capacity. Average costs increase significantly as flow rate falls below design capacity. Because the system must be able to handle the peak flows which occur only occasionally, much of the time there is idle capacity resulting in higher average costs. The degree of the idle capacity is directly related to the variation in the flow rate of the influent. For example, resort communities which may experience a population influx of many times the resident population have found average costs during the off season excessive. Also, systems which have combined sewer and storm drains enjoy lower construction costs, but the burden on the treatment plant during the rainy season often requires an increased capacity which lies idle during the dry months.

The next greatest influence upon the variation observed in the cost data comes from topographical/environmental/health constraints. This factor contributes almost 20 percent of the observed variation in the study group. Underlying this factor are determinants such as terrain, soil characteristics, nature of the ecosystem, and safeguards necessary to prevent contamination. An example of a costly area, in terms of this factor, would be one which has irregular terrain, rocky soil conditions, a delicately balanced ecosystem, and a shallow aquifer. Alternatively, an urban environment may be thought of as an example of a costly area due to the high premium paid for land, the population density, and the ease of contagion.

The third factor contributed approximately 10 percent of the observed variation and is directly related to the sludge handling process. In the study group we observed combinations of sand bed, lagoon, vacuum, and centrifuge dewatering processes with incineration, land fill, land reclamation, soil condition, and ocean discharge disposal methods.

The fourth factor identified contributed nearly 10 percent of the variation observed in the study group, This factor represents the selection and the configuration in which the train of wastewater treatment unit processes are organized.

Spray Irrigation Distribution

The influencing factors were more well defined for this subdivision of the system, engineering design parameters being responsible for all the observed variation in the study group. The first factor which accounts for 41.5 percent of the variation is the dynamic hydraulic head. The second factor, accounting for 26 per cent of the observed variation, is the irrigation system capacity. The last factor accounted for roughly 16 percent of the variation and represents the distance for conveyance to the irrigation site. These three factors are all related by the design parameters: sprinkler spacing, application rate, nozzle size and pressure, and the loading potential of the irrigation site. Discussion of design criteria may be found in Pair (11) and computer algorithms for devising least cost sprinkler irrigation systems are available. There appears to be little room for further optimizing the system configuration in this subsection. Most of the engineering criteria are dictated by the factor which has been interpreted as "topographical/environmental/health constraints."

NON-MONETARY COSTS

With the recent and predicted energy costs escalation there appears to be a continuing trend of definite cost advantages associated with land renovation, but non-monetary costs in terms of land quality must be evaluated. Some soil degradation with prolonged wastewater application appears to be inevitable. The nature of this degradation will depend upon the quality of the effluent and the original soil characteristics.

Effluent irrigation studies over a six-year period, Youngner et. al., (12), have shown that certain ions will accumulate in the soil. If sodium, common to most effluent waters, is adsorbed on clay particles in large amounts is will have a destructive effect on soil structure. As the SAR (sodium adsorption ratio) approaches 9 or higher, soil permmeability problems may be anticipated. With a loss of permeability, the soil will function poorly as a wastewater renovator and as medium for plant growth. High effluent bicarbonate levels will further increase the sodium hazard. Although a high SAR can be corrected by the addition of calcium, this will add to the total cost of the recycling process.

Increased salinity and sometimes high boron levels may also be anticipated after several years of application unless amounts of water are adequate for good leaching. Again, this can often be corrected but at an additional cost.

Heavy metals, present in many effluent waters, may also accumulate in amounts which could be harmful to plant growth or could prevent future use of the land for food production. The heavy metals most often found in municipal wastewater are copper, cadmium, nickle and zinc. Fortunately, during secondary treatment a high proportion of these elements are removed in the sludge, Peterson et al., (13).

None of these problems appear to be of sufficient magnitude to cause serious soil degradation or to render the land useless. Therefore, land degradation should seldom if ever be a factor preventing the recycling of wastewater by spray irrigation.

CONCLUSION

Land application renovation of pretreated wastewater is cost effective in producing water quality which is comparable to other methods of advanced wastewater treatment.

The area of greatest potential savings is the degree of system capacity utilization. An analysis of alternative strategies for utilizing facilities to their full design capacities could recommend policies producing appreciable savings. For example, i) incentives/penalties could be implemented for system users during the slack/peak flow periods, ii) holding areas could be utilized to maintain a constant capacity flow into the treatment system, or iii) some combination of i, ii, and regulatory controls might be used.

The method of sludge handling in conjunction with the pretreatment system process train was significant in explaining the observed cost variations in the study group.

The land application sub-system of the renovation process was the most stable cost of the total system design and lends itself readily to simulation analysis for determining optimal configurations in alternative settings.

Acknowledgment

This research was conducted with support from the University of California Water Resources Center (project No. W-532) and the United States Office of Water Research and Technology (project No. A-065-CAL).

The authors would like to thank the consulting engineers, municipal administrations, construction firms, and the California Water Resources Control Board whose cooperation made this report possible.

REFERENCES

(1)Van Note, Robert H.; Paul V. Herbert; Ramesh M. Patel; Craig Chupek;& Lester Feldman. A Guide to the Selection of Cost-Effective Wastewater Treatment Systems. U. S. Environmental Protection Agency, Washington D. C. (1975).

(2)Middleton, F. M. "Systems & Costs for Water Renovation & Reuse" Water Renovation & Reuse (Hillel I. Shuval ed.), Academic Press, San Francisco (1977).

(3)Pound, Charles E.; Ronald W. Crites;& Douglas A. Griffes. Costs of Wastewater Treatment by Land Application. U. S. Environmental Protection Agency, Washington D. C. (1975).

(4)Young, C. Edwin. Land Application of Wastewater: A Cost Analysis. U. S. Department of Agriculture, Washington D. C. (1978).

(5)The Energy Task Force, California Water Resources Control Board. Effects of Energy Shortage on the Treatment of Wastewater in California. California Department of Water Resources, Sacramento (1974).

(6)California Water Resources Control Board. Treatment Processes for Wastewater Reclamation for Ground Water Recharge. California Department of Water Resources & Department of Health (1975).

(7)PRC Toups Corporation;& James M. Montgomery Consulting Engineers. Draft Summary Report for Wastewater Reclamation and Conveyance Facilities. Mimeograph (1978).

(8)EPA Research Center. Electric Power Consumption for Municipal Wastewater Treatment in the United States. U. S. Environmental Protection Agency, Washington D. C. (1973).

(9)Dames & Moore, Water Pollution Control Engineering Services. Analysis of Operations and Maintenance Costs for Municipal Wastewater Treatment Systems. U. S. Environmental Protection Agency, Washington D. C. (1978).

(10)Fuller, W. A. & G. E. Battese. "Estimation of Linear Models With Crosses-Error Structure" Journal of Econometrics. 2:1:67 (May 1974).

(11)Pair, C. H. (ed.). Sprinkler Irrigation. Sprinkler Irrigation Association, Washington D. C. (3rd edition 1969) (supplement 1973).

(12)Youngner, V. B.; T. E. Williams;& S. R. Green. Ecological and Physiological Implications of Greenbelt Irrigation. California Water Resources Center Contribution No. 157, University of California, Davis (April 1976).

(13)Peterson, J. R.; Cecil Lue-Hing;& D. R. Zenz. "Chemical and Biological Quality of Municipal Sludge" Recycling Treated Municipal Wastewater and Sludge Through Forest and Cropland (W. E. Sopper and L. T. Kardos, eds.). Pennsylvania University Press (1973).

BIBLIOGRAPHY

BRADLEY, R. M. & PERTER C. G. ISSAAC. Capital & Operating Costs of Sewage Treatment. University of Newcastle Upon Tyne, Oriel Press, (1969)

BRANSON, ROY & ROBERT AYERS. Using Reclaimed Water On Farmland. University of California, Cooperative Extension (July 1977).

BUCHALTER, HOWARD. Land Use Alternatives Using Input-Output Analysis. U. C. Santa Barbara, Research Reports in Public Policy No. 5, (April 1976).

CALIFORNIA DEPARTMENT OF HEALTH. Reliability of Wastewater Reclamation Facilities. Department of Health, Sacramento (1976).

CALIFORNIA DEPARTMENT OF WATER RESOURCES. Treatment Processes for Wastewater Reclamation for Ground Water Recharge. California Department of Water Resources & Department of Health (1975).

CALIFORNIA DEPARTMENT OF WATER RESOURCES. Wastewater Reclamation, State of the Art. Department of Water Resources Bulletin No. 189 (March 1973).

DALTON, FRANK E. & RICHARD R. MURPHY. "Land Disposal IV: Reclamation & Recycle" Jour. WPCF, 45:7:1489 (July 1973).

EPA. A Guide to the Selection of Cost-Effective Wastewater Treatment Systems. Washington D. C., U. S. Government Printing Office (July 1978).

EPA. Analysis of Operation & Maintenance Costs for Municipal Wastewater Treatment Systems. Washington D. C., U. S. Government Printing Office (May 1978).

EPA. Bibliography of Water Pollution Control Benefits and Costs. Washington D. C., U. S. Government Printing Office (October 1974).

EPA. Construction Costs for Municipal Wastewater Conveyance Systems; 1973-1977. Washington D. C., U. S. Government Print Office (May 1978).

EPA. Cost Analysis of Water Pollution Control: An Annotated Bibliography. Washington D. C., U. S. Government Printing Office (April 1973).

EPA. Cost of Wastewater Treatment by Land Application. Washington D. C. U. S. Governemtn Printing Office (June 1975).

EPA. Land Application of Sewage Effluents and Sludges: Selected - Abstracts. Washington D. C., U. S. Government Printing Office (June 1974).

EPA. Survey of Facilities Using Land Application of Wastewater. Washington D. C., U. S. Government Printing Office (July 1973).

EPA. The Economics of Clean Water (vol. 1,2, & summary). Washington D. C., U. S. Government Printing Office (1972).

EGELAND, DUANE R. "Land Disposal I: A Giant Step Backward" Jour. WPCF. 44:1:1 (July 1973).

EVANS, DAVID R. & JERRY C. WILSON. "Capital & Operating Costs--AWT" Jour. WPCF. 44:1:1 (January 1972).

"EPA Seqage Regs 'Waste Energy'" Engineering News Record. 201:7:9 (August 17, 1978).

FULLER, W. A.;& G. E. BATTESE. "Estimation of Linear Models with Crosses-Error Structure" Journal of Econometrics. 2:1:67 (May 1974).

GORSUCH, RICHARD L. Factor Analysis. W. B. Saunders Co., Philadelphia (1974).

JOHNSON, JAMES F. Renovated Wastewater, an Alternative Source of Municipal Water Supply in tne United States. University of Chicago, Department of Geography Research Paper No. 135 (1971).

KASPERSON, ROGER E. & JEANNE X. KASPERSON. Water Re-Use & the Cities. University Press of New England, Hanover, New Hampshire (1977).

KNEESE, ALLEN V. & BLAIR T. BOWER. Managing Water Quality: Economics, Technology, Institutions. John Hopkins Press, Baltimore (1968).

LAWLEY, D. N. & A. E. MAXWELL. Factor Analysis As A Statistical Method. Butterworth & Co., London (2nd ed., 1971).

MACRAKIS, MICHAEL S. (ed.). Energy: Demand, Conservation, and Institutional Problems. The M. I. T. Press, Cambridge, Mass. (1974).

ORCUTT, RICHARD G. An Engineering-Economic Analysis of Systems Utilizing Storage for the Irrigation of Parks and Golf Courses with Reclaimed Wastewater. Desert Research Institute, University of Nevada, Reno (1967)

PAIR, C. H. (ed.). Sprinkler Irrigation. Sprinkler Irrigation Association, Washington D. C. (3rd edition 1969) (Supplement 1973).

PRC TOUPS CORPORATION & JAMES M. MONTGOMERY CONSULTING ENGINEERS. Draft Summary Report for Wastewater Reclamation and Conveyance Facilities Mimeograph (December 1978).

SEARL, M. (ed.). Energy Modeling. Resources for the Future, Washington D. C. (1973).

SHAH, KANTI, L. & GEORGE W. REID. "Techniques for Estimating Construction Costs of Waste Treatment Plants" Jour. WPCF. 42:5 par 1:776 (May 1970).

SHUVAL, HILLEL I. (ed.). Water Renovation and Reuse. Academic Press, New York (1977).

SOPPER, W. E. & L. T. KARDOS (eds.). Recyclinq Treated Municipal Wastewater and Sludge Through Forest and Cropland. Pennsylvania State University Press (1973).

THE ENERGY TASK FORCE, CALIFORNIA WATER RESOURCES CONTROL BOARD. Effects of Energy Shortage on the Treatment of Wastewater in California. California Department of Water Resources, Sacramento (1974).

THOMAS, RICHARD E. "Land Disposal II: An Overview of Treatment Methods" Jour. WPCF. 45:7:1476 (July 1973).

"Town Abandons Tertiary Sewage Plant" Engineering News Record. 201:1:11 (July 6, 1978).

U. S. COMMISSION ON POPULATION GROWTH AND THE AMERICAN FUTURE. Population, Resources and the Environment. Washington D. C., U. S. Government Printing Office (1972).

YOUNG, C. EDWIN. Land Application of Wastewater: A Cost Analysis. U. S. Department of Agriculture, Washington D. C. (1978).

YOUNGNER, V. B.; T. E. WILLIAMS;& L. R. GREEN. Ecological and Physiological Implications of Greenbelt Irrigation. California Water Resources Center Contribution No. 157, University of California, Davis (April 1976).

FIGURE 1

THE STUDY AREA

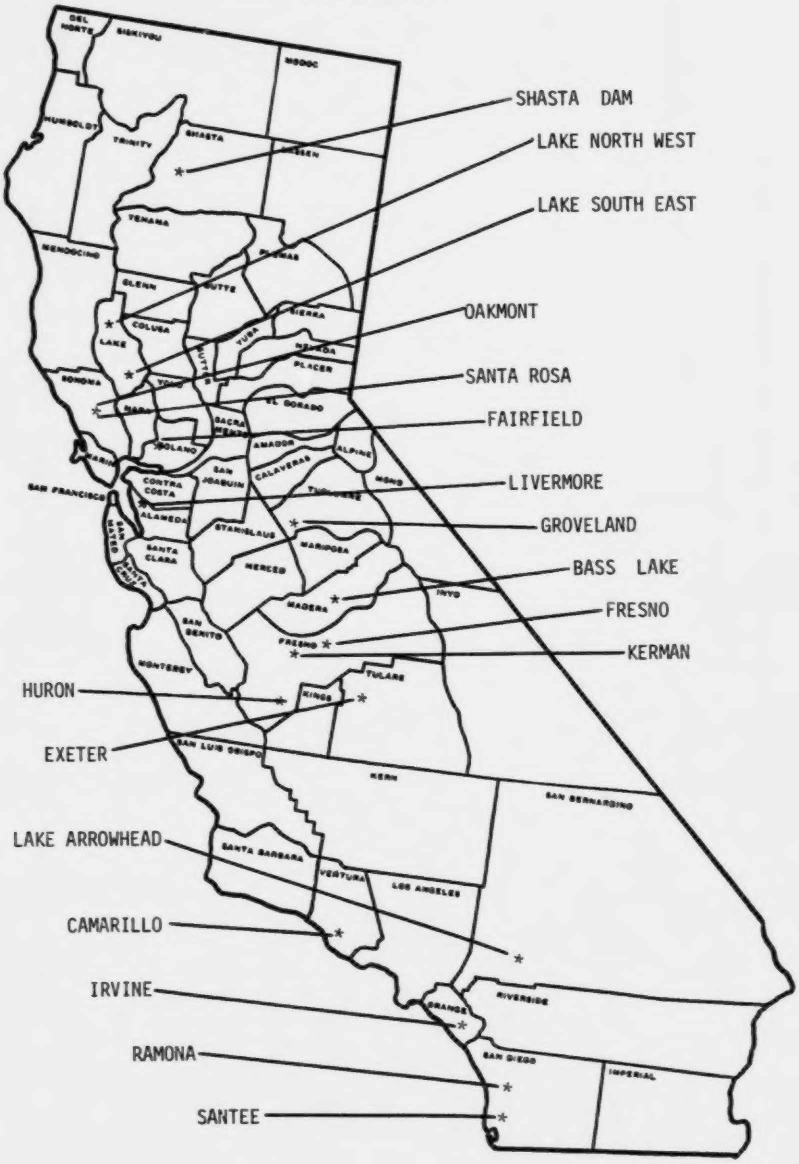


TABLE 1
Percentage Distribution Of The Study Group By Type System And Topographical Setting

TYPE SYSTEM	TOPOGRAPHICAL SETTING Mountainous	 Rolling Hills	 Valley	 Plains	Row Percent Total
Activated Sludge	11.1	5.6	16.7	0	33.3
Extended Aeration	0	5.6	0	5.6	11.1
Activated Sludge with Extended Aeration	0	5.6	5.6	0	11.1
Activated Sludge with Waste Stabilization Pond and Perculation Bed	0	5.6	0	0	5.6
Activated Sludge with High Rate Trickling Filter	0	0	5.6	0	5.6
Activated Sludge with Contact and Extended Aeration	5.6	0	0	0	5.6
Activated Sludge with Coagulation Filter	0	5.6	0	0	5.6
Activated Sludge with High Rate Trickling Filter and Tertiary	0	0	5.6	0	5.6
Waste Stabilization Pond	0	0	0	5.6	5.6
Other	0	11.1	0	0	11.1
Column Percent Total	16.7	38.9	33.3	11.1	

Raw Chi-Square = 30.78568 with 27 degrees of freedom. Significance = 0.28.

TABLE 2

Percentage Distribution of the Study Group by Type Treatment Plant and Distribution System

TYPE SYSTEM	DISTRIBUTION SYSTEM Agriculture	Golfcourse	Greenbelt	Other	Row Percent Total
Activated Sludge	16.7	5.6	5.6	5.6	33.3
Extended Aeration	11.1	0	0	0	11.1
Activated Sludge with Extended Aeration	11.1	0	0	0	11.1
Activated Sludge with Waste Stabilization Pond and Perculation Bed	11.1	0	0	0	11.1
Activated Sludge with High Rate Trickling Filter	5.6	0	0	0	5.6
Activated Sludge with Contact and Extended Aeration	0	0	0	5.6	5.6
Activated Sludge with Coagulation Filter	0	0	0	5.6	5.6
Activated Sludge with High Rate Trickling Filter and Tertiary	5.6	0	0	0	5.6
Waste Stabilization Pond	0	0	0	5.6	5.6
Other	11.1	0	0	0	11.1
Column Percent Total	61.1	5.6	5.6	27.8	100.0

Raw Chi-Square = 18.54544 with 27 degrees of freedom. Significance = 0.8860.

TABLE 3

Percentage Distribution of the Study Group by Distribution System and Topographical Setting

Distribution System	TOPOGRAPHICAL SETTING Mountainous	 Rolling Hills	 Valley	 Plains	Row Percent Total
Agriculture	5.6	22.2	27.8	5.6	61.1
Golfcourse	0	5.6	0	0	5.6
Greenbelt	5.6	0	0	0	5.6
Other	5.6	11.1	5.6	5.6	27.8
Column Percent Total	16.7	38.9	33.3	11.1	100.0

Raw Chi-Square = 8.15064 with 9 degrees of freedom. Significance = 0.5190.

* * *

TABLE 4

Factor Analysis Results Using Alpha Factor and Varimax Rotation

COLLECTION/PROCESSING

Factor	Eigenvalue	% of Variation	Cum %
(1) Capacity	5.40417	45	45
(2) Topographical/ Environmental/ Health	2.37856	19.8	64.8
(3) Sludge Handling	1.23782	10.3	75.1
(4) Process Train	1.1348	9.5	84.6

DISTRIBUTION/IRRIGATION

Factor	Eigenvalue	% of Variation	Cum %
(1) Hydraulic Head	3.2269	41.5	41.5
(2) Capacity	2.07713	26	67.5
(3) Conveyance Distance	1.30007	16.3	83.7

TABLE 5

ESTIMATED RELATIONSHIPS

(t statistics in brackets)

[1]TOTAL ENERGY = -290 + 1107 [2]RATED CAPACITY
[8.76]* R^2 = .638

TOTAL ENERGY = 981 + 1787 [3]INFLOW
[8.69]* R^2 = .825

[4]TOTAL COST = 6,054,325 + 572 TOTAL ENERGY
[3.7]* R^2 = .462

[5]O & M = 312,454 + 43,584 RATED CAPACITY
[5.0]* R^2 = .638

TOTAL ENERGY = -2178 + 5.283 [6]TOTAL HORSEPOWER
[3.1]* R^2 = .549

O & M = 107,607 + 209,357 INFLOW
R^2 = .868

TOTAL COST = 5,348,907 + 1,348,060 INFLOW
[5.6]* R^2 = .663

TOTAL COST = 4,562,976 + 811,796 RATED CAPACITY
[5.1]* R^2 = .635

* Significant at the .005 level.

[1] In Millions of British Thermal Units
[2] In Units of Million Gallons per Day
[3] In Units of Million Gallons per Day
[4] In November 1978 Dollars
[5] In November 1978 Dollars
[6] In Units of One Horsepower

TABLE 6

Distribution of Annual Average Operations & Maintenance Cost per MGD Flow by Type of Sludge Disposal

Average O&M Cost

Disposal Type	Low ($3,300)																	High ($1,726,000)
Incineration																	X	
Land Fill								X										
Land Reclamation				X		X							X		X	X		X
Soil Conditioner		X	X								X	X						
Other	X				X		X		X	X				X				

Each entry represents one system's placement in the low to high range.

****

TABLE 7

Distribution of Annual Average Operations & Maintenance Cost per MGD Flow by Type of Sludge Process

Average O&M Cost

Process Type	Low ($3,300)																	High ($1,726,000)
Sand Beds	X		X	X	X				X		X	X	X		X	X		X
Lagoons						X		X										
Vacuums																	X	
Centrifuge										X								
Other		X					X							X				

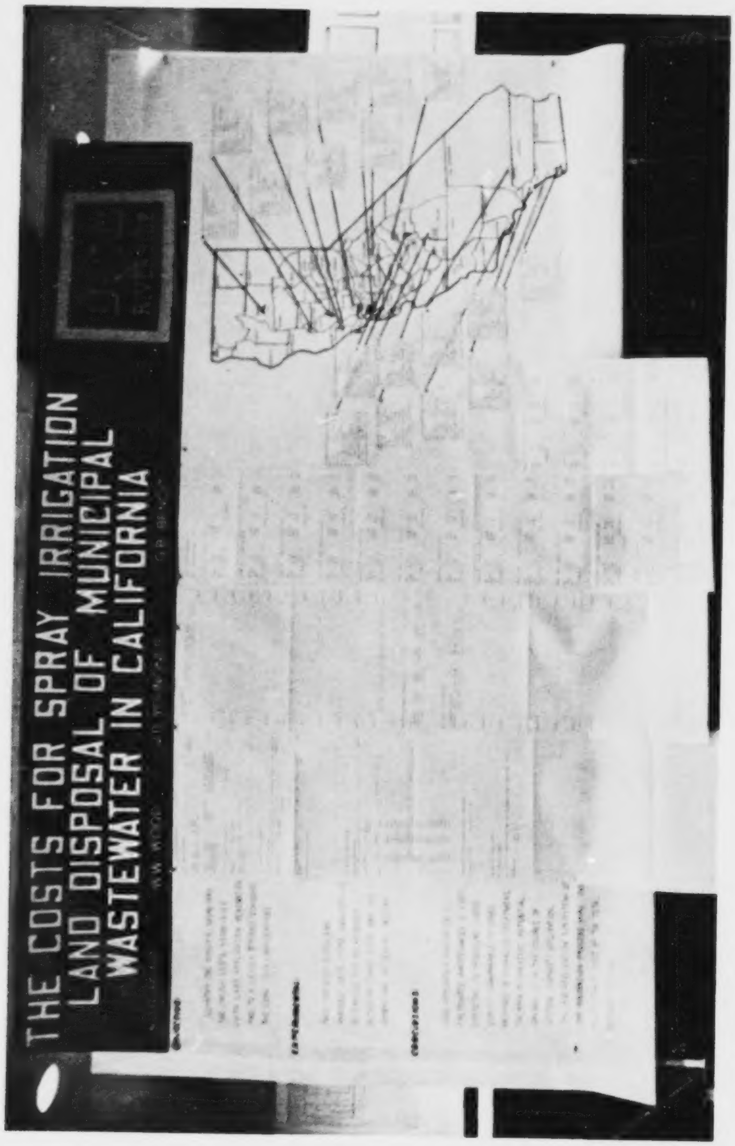
Each entry represents one system's placement in the low to high range.

TABLE 7

Distribution of Annual Average Operations & Maintenance Cost per MGD Flow by Type of Treatment System

Treatment Type	Low ($3,300)									Average O&M Cost							High ($1,726,000)
Activated Sludge							X	X	X	X					X	X	
Extended Aeration		X			X												
Activated Sludge with Extended Aeration											X			X			
Activated Sludge with Waste Stabilization Pond and Perculation Bed													X				
Activated Sludge with High Rate Trickling Filter			X														
Activated Sludge with Contact and Extended Aeration																	X
Activated Sludge with Coagulation Filter										X							
Activated Sludge with High Rate Trickling Filter and Tertiary												X					
Waste Stabilization Pond	X																
Other				X		X											

Each entry represents one system's placement in the low to high range.



THE COSTS FOR SPRAY IRRIGATION LAND DISPOSAL OF MUNICIPAL WASTEWATER IN CALIFORNIA

W.W. WOOD

EFFLUENT REUSE IN A SEMI-ARID REGION OF THE UNITED STATES

Gary R. Windolph
Vice President, ARIX Corporation
(Formerly C-E Maguire, Inc.)

Timothy J. Carlson
Project Engineer, ARIX Corporation
(Formerly C-E Maguire, Inc.)

Essential to the management of wastewater is the method of final disposal of the effluent. Historically, treated wastewater was discharged to the nearest body of water with no thought of recovering nutrients or recycling the water. That philosophy has changed in recent years as an outgrowth of public concern with the waste of this country's valuable resources.

The City of Sterling, Colorado has provided wastewater treatment services to the residents of that city since 1937. The initial treatment plant which provided primary treatment was upgraded to a secondary plant in 1961. This plant was expanded in 1966 to create additional capacity for treatment of waste from an enlarged industrial discharge from the Sterling Beef Company. Since that time, performance and effluent quality of the treatment plant have come under close scrutiny from the State Health Department and the Federal Environmental Protection Agency because of consistent violations of permit conditions. In July, 1977, the City of Sterling adopted a plan to upgrade their treatment capacity which included the abandonment of the existing plant with the wastewater to be conveyed to a new aerated lagoon system located four miles northeast of the City. The siting of the new lagoon system was a prime consideration in the selection of this plan, because this site afforded many opportunities to reuse the treated wastewater. As a condition of the adoption of this plan, an in-depth study was to be made of the land application and/or the reuse potentials of the lagoon effluent. This was to be done concurrently with the design of the lagoon and transmission facilities. The scope of the study envisioned several general land application alternatives for treatment and reuse of lagoon effluent. Basic options considered as alternatives were:

1. Discharge to the South Platte River.
2. Land application on existing agricultural land via existing irrigation ditches in the vicinity of the treatment lagoon.
3. Land application for ground water recharge.
4. Land application for crop irrigation.
   a. Utilization of year round effluent.
   b. Utilization of summer effluent only.
5. Delivery of winter discharges for wetlands development.
6. Sale of effluent.
   a. Summer effluent only.
   b. Year round effluent.

STUDY AREA

The City of Sterling, Colorado is located 125 miles northeast of Denver, Colorado, and is the largest city between Denver and North Platte, Nebraska. The City is located in one of the State's richest agricultural and oil production areas. Situated on flat terrain in the South Platte valley, the community is surrounded by a productive, irrigated farming region which produces large quantities of corn, wheat and sugar beets. In addition, the agricultural segment of the economy also includes many cattle feeding and dairy production farm units. The area has a semi-arid, continental climate with an abundance of sunshine throughout the year. The total annual average precipitation is 14.7 in. while the average annual temperature is 48.9° Fahrenheit. The frost free period for Sterling is on the order of 140 days.

The general area studied for the reuse and disposal alternatives is indicated in Figure 1. The South Platte River forms the western and northern boundary. Lands west of the river in the vicinity of the new treatment site were not considered because they are fully utilized as irrigated farmland with the exception of the river bottom.

Wastewater flow projections used in the analysis of alternative disposal and reuse plans are presented in Table 1.

WATER QUALITY CRITERIA

Prerequisite to the development and evaluation of disposal or reuse alternatives is the establishment of the water quality criteria pertinent to each alternative. The Colorado Department of Health, which is the authority for developing water quality criteria, defined effluent quality requirements for each of Sterling's disposal/reuse options. The established requirements are summarized as follows for discharges from the 3.88 mgd three celled aerated lagoon system:

1. Direct discharge to the South Platte River or to an irrigation ditch used to irrigate private farm land would have an effluent quality requirement for
    a. Biochemical oxygen demand (BOD) of 30 mg/l for the monthly average and 45 mg/l for the weekly average.
    b. Total suspended solids (TSS) of 30 mg/l for the monthly average and 45 mg/l for the weekly average.
    c. Fecal coliform count of 6,000 per 100 ml monthly geometric mean and 12,000 per 100 ml weekly geometric mean.

2. Discharge to City owned or leased land application site or for ground water recharge, less stringent requirements but would be evaluated on a case-by-case basis.

3. Discharge used to develop wetlands would probably require a BOD of 30/45 mg/l, TSS of 30/45 mg/l, and fecal coliform of 6,000/12,000 per 100 ml. This could be reduced depending upon what classification, if any, is assigned to the wetland ponds which were to be developed.

Based upon pilot study data, Sterling's future aerated lagoon system will produce BOD concentrations of less than 30 mg/l and Fecal coliform counts of less than 6,000 per 100 ml for both warm and cold weather operations. Disinfection will be achieved by sufficient detention time in the lagoons rather than by direct means such as chlorination. TSS, which is associated with algae production in the lagoons, may be greater than 30 mg/l for some portions of the year. It was therefore apparent that for Sterling's system, "less stringent requirements" only applied to TSS concentrations which could be higher than 30 mg/l when applied to city owned or leased agricultural lands and for ground water recharge. It was likely that this same criteria would hold for wetlands if they were confined systems with no outlets to other bodies of water. If the ponds were flow through systems, the more stringent criteria would apply.

Although aerated lagoon effluent meets all public health standards for the irrigation of agricultural land and for ground water recharge, there are other constituents of the wastewater which would impact reuse alternatives. These include nitrates, salinity, dissolved sodium, acidity, primary nutrients, trace elements, viruses and bacteria.

Nitrates

The US Public Health Service recommends that the Nitrate (NO_3-N) concentration should not exceed 10 mg/l in the public water supply. From the pilot study data on the Sterling aerated lagoon system, NO_3-N concentrations will be highest during the summer at approximately 14 mg/l and near zero levels during the colder winter months. This is not to say that the NO_3-N concentrations in the soil will be the same. As the wastewater passes through the soil, Ammonia-N can be nitrified to NO_3-N while NO_3-N can be denitrified to nitrogen gas. Because of these chemical reactions within the soil mantle, the ground water NO_3-N concentration cannot be specifically defined; however, due to high dilution between percolated wastewater and existing ground water there should not be any violations of drinking water standards. To verify this, all alternatives which might affect ground water quality must include an extensive ground water monitoring system.

Salinity

When production of crops is a goal of land application, the concentration of dissolved salts in irrigation water may impact crop production. The salts concentration in pilot plant effluent was found to range between 2,780 and 2,870 mg per liter. This is equivalent to a specific conductance of 4,340 umho per centimeter. As a comparison, the average specific conductance of the South Platte River is 1,700 umho per centimeter.

The US Salinity Laboratory has classified irrigation waters as to their salinity hazard. Under this system, the river water is classified as having high salinity while the wastewater is considered as having very high salinity. As a result of the South Platte's high salinity, some limited problems with salt buildup have occurred in the relatively heavy soils near the river.

While production of crops is not precluded by irrigating with Sterling's wastewater, certain conditions and precautions are recommended for successful crop growth. The soil should be permeable with adequate drainage, low salt tolerant crops should be avoided and water in excess of that needed by the crop must be provided. The latter requirement of applying excess irrigation water to drive accumulated salts below the crop root zone is referred to as leaching. Leaching requirements for growing corn and alfalfa with Sterling's wastewater effluent was calculated to be 0.4. These crops are classified as being moderately salt tolerant. As a result of extensive analysis, it was determined that the use of Sterling's effluent at a 0.4 leaching fraction will cause a reduction of field corn production of twenty per cent and alfalfa production of ten per cent. Although a 0.4 leaching requirement may be considered conservative to some, analysis of a reduced leaching fraction showed that the relatively small irrigation acreage added, on the whole did not compensate for the additional overall reduction in yields.

Dissolved Sodium

High dissolved sodium concentrations in irrigation waters may produce sodic conditions in the soils. This condition can produce detrimental effects on crop nutrition and also reduce the permeability of the upper soil profile. Clay soils are more susceptible to developing sodic conditions than soils with low clay content such as are considered for irrigation in this area. Also, problems with sodicity are much less likely to develop with the use of high salt content irrigation water. Sodicity is not believed to be a concern in the use of Sterling's wastewater for crop irrigation. Concentrations of sodium and chlorides in water applied by sprinkler irrigation may cause toxic accumulations in the leaves, but this problem is mainly limited to the fruit crops and is not a problem with corn or alfalfa.

Acidity

The wastewater effluent of the pilot plant had an average pH of 8.2, or somewhat alkaline. This pH value does not present a problem for any of the alternatives studied.

Primary Nutrients - Nitrogen, Phosphorus and Potassium

Except for potassium, the concentration of these nutrients in the wastewater is insufficient to provide the recommended levels for crop production of corn and alfalfa without the addition of commercial fertilizers. While the potential exists for nitrogen percolation to the ground water due to the leaching required to control salinity, the careful scheduling of leach water additions to minimize loss of nutrients is expected to reduce the chances for a problem from the irrigation sites.

Trace Elements

Concentration of trace elements from Sterling's wastewater was not assessed to have detrimental effects either on the ground water or plant growth if used for land application. Trace element concentrations found in the wastewater did not exceed drinking water standards or threshold limits for plant toxicity.

Viruses and Bacteria

Although the number of viruses was not tested in the effluent, the experience from numerous EPA Pilot Studies indicates a substantial virus reduction can be realized through land application methods. Bacteria reductions naturally occurring with the lagoon system are expected to meet Colorado Department of Health standards.

ALTERNATIVE DEVELOPMENT

Selection of the site for the new sewage treatment lagoon provided for the development of eighteen different alternatives for ultimate disposal and reuse. The eighteen alternatives can be divided into six basic categories:

- Discharge to the South Platte River
- Land application on existing agricultural land
- Land application on new agricultural land
- Ground water recharge
- Discharge for wetlands development
- Sale of treated wastewater for agricultural use by others

Each of these potential basic alternatives was conceptually developed and evaluated. Basic alternatives are shown in Figure 2.

Discharge to the South Platte River

Direct discharge is a pure disposal alternative with no direct reuse benefits. For the South Platte River, which is fully appropriated during the summer months, it is obvious that downstream irrigators do make use and successive use of the water and its nutrient values. Thus it might be said that even direct discharge constitutes at least an indirect reuse of the effluent.

Effluent quality criteria required that a TSS removal system be constructed following the aerated lagoon process. Analysis of several removal processes was performed to find the most effective system. Intermittent sand filtration was found to be the best TSS removal process and thus was used for this alternative and subsequent ones that require TSS removal. Chlorination facilities were not required because the detention time of the aerated lagoon system was sufficiently long to afford a natural reduction in fecal coliform organisms.

Land Application on Existing Agricultural Land

This alternative was specific to the Bravo Ditch system and its associated irrigated lands. Both irrigation season and non-irrigation season discharges to the ditch were analyzed. Irrigation season discharges to the ditch required sand filtration to meet effluent TSS criteria. In order for downstream appropriators not to be injured, the effluent would merely replace South Platte River water and not be an additional water supply. Because the historic water flows of the ditch would not be increased, no additional cost for ditch maintenance or phreatophyte control would be necessary. It was concluded that the winter discharge to the ditch was not feasible because of potential icing problems during winter operation.

Land Application on New Agricultural Land

The land application of lagoon effluent on agricultural land has the benefits of TSS and nutrient removal through crop production. Because land application is viewed as a treatment process for TSS removal, EPA guidelines require that the City own or lease the agricultural site. The availability of land then became one of the criteria for choosing land application sites. Other criteria considered included soil types best suited for highly saline water, soil types which have optimum crop production potential, land slope, location close to the treatment plant with sufficient contiguous suitable land, surrounding land use, flood plain considerations, and geology. Based on these criteria, three suitable land applications were identified for further study. It is appropriate at this time to point out, that while the City must own or lease the land application site, it does have the option of operating the site with City work forces, having the site custom farmed or leasing the site to be farmed by private individuals.

Using optimum farming practices for each of the three sites, it was found that with winter storage, five center pivot systems irrigating 126 acres each would fully utilize effluent flows between 1980 and 1988. In 1988, two more pivots would be added, with two more in 1996. Crops would consist of corn, and/or alfalfa, grown in proportion to optimize the use of the wastewater. For alternatives which do not include winter storage, three center pivots would initially be used in 1980, with one added in 1988 and one more in 1996.

As with all commodities, the crop market can be the determining factor as to the feasibility of an alternative land application system. To assess the reliance of land application alternatives on the market price of crops, a sensitivity analysis was prepared. This analysis showed that no relative change occurred between alternate costs based upon varying market prices of the crops grown.

Part of the cost of any land application system in a semi-arid area where water has a high value is the cost of water rights to augment new consumptive use. In the Sterling area, the cost per acre foot of consumptively used water is approximately $1,700. Consumptive use is a function of the type of crop grown, the irrigation site, and the amount of leach water returned to the river during the irrigation season. Consumptive uses for each irrigation site were analyzed and the cost for supplemental water was included in the cost for each alternative. For alternatives which store winter effluent there is a potential that the storage rights to the downstream Julesburg Reservoir could be injured in years of low flow. To prevent this injury, the City would purchase storage rights or provide a partial sand filter to bypass flows back to the river when a call is made by senior water rights. Analysis of these two contingency options indicated that the provision of a partial sand filter was more cost effective than standby storage water rights; thus, all land application alternatives with winter storage include the cost for a partial sand filter.

Storage is an integral part of all land application systems. Winter storage provides the detention needed for both non-irrigation season flows and flows during the irrigation season which exceed crop requirements. Summer storage is needed to store flows which exceed crop requirements. Storage for the alternatives studied could be provided by a natural depression located within the area of the treatment plant. With only minor earthwork, a 2,000 acre foot reservoir could be constructed at this location. A 500 acre foot reservoir for summer storage only could be constructed adjacent to the treatment plant.

Ground Water Recharge

The concept of ground water recharge is to apply non-irrigation season water to the ground water aquifer at a discharge location which will cause the recharge water to return to the river during the summer months, thereby gaining water rights for the City. If the recharge point is too close to the river, the bulk of the recharge water will get to the river too soon. If the recharge point is too far from the river, it will take many years for water to return to the river. To locate the ideal recharge site, five suitable ground water recharge locations were analyzed using the Colorado Division of Water Resources computer ground water modeling program. This analysis showed that the best site was a natural depression one mile south of the treatment plant. The amount of new water or new summer water rights gained by the City under this alternative would average approximately 800 acre feet annually during the planning period. To provide for occasional winter river calls by the downstream Julesburg Reservoir, effluent could be bypassed through a partial sand filter back to the South Platte River at times of call.

Discharge for Wetlands Development

At the time of initiation of the study, the Colorado Division of Wildlife was seriously considering developing a new wildlife wetlands area east of Sterling's treatment site which would have used treated effluent. Subsequently, during development of the alternatives, the Division of Wildlife altered its plans and began actively considering purchase of property within the South Platte River flood plain. For the plan to provide reuse water for wildlife enhancement and wetlands development, sand filtration is needed since the pond system to be developed would be of a flow through nature with all overflows directed back to the river. Since no consumptive use of the water would take place during the winter months when the system is in operation, there was not a need to provide for augmentation water.

Sale of Treated Wastewater for Agricultural Use

Direct sale of reuse water requires that the effluent meet secondary treatment standards and that an augmentation plan be instituted to offset consumptive use to prevent injury to downstream water rights. Secondary treatment is required because the City must be responsible for the treatment of the wastewater and, if it is sold loses control of the ability to insure the necessary level of treatment.

ECONOMIC ASSESSMENT OF THE ALTERNATIVES

To evaluate any disposal or reuse system it is necessary to evaluate the cost involved to construct and operate these systems. One manner of dealing with this issue is to compare the equivalent annual cost of each alternative. The equivalent annual cost is simply the amortized capital cost plus the annual operation and maintenance costs and the salvage value of the system at the end of the planning period. The equivalent annual cost is a costing procedure used by EPA to define the most cost effective alternative for funding purposes. By EPA guidelines, revenue from land application systems, i.e. crop sales, can be credited to help offset the cost of the overall system. However, water supply benefits such as the production of additional raw and domestic water sources, cannot be credited. Thus, when dealing with the cost effective analysis of reuse alternatives by EPA criteria, the City does not receive a realistic comparison of alternatives when water supply benefits are a part of any alternative. To correct this discrepancy while at the same time satisfying EPA criteria, two complete sets of cost analysis were developed for purposes of economic analysis. The first, which is called the "Equivalent Annual Cost," uses the EPA approach. The second analysis, called "Average Annual Cost to the City," shows the average annual cost which Sterling would incur over the twenty year life of the project, accounting for both water supply credits and inflation. The Equivalent Annual Cost analysis accounts for neither of these factors. Costs from the two approaches obviously differ, but were useful in helping EPA determine the eligibility of the project funding while aiding the City in their determination of what is the least cost alternative to them.

The Equivalent Annual Cost and the Average Annual Cost to the City, along with the total capital cost and average annual O&M cost of each of the alternatives, are summarized in Table 2. It should be noted that non-economic factors were also taken into consideration when analyzing the alternatives.

THE SELECTED PLAN

The plan selected for implementation by the City of Sterling consists of a system providing for direct discharge to the South Platte River during summer months, with discharge to a ground water recharge site during the winter months. Water that is discharged to the South Platte River during the summer months requires facilities for TSS removal. As discussed previously, the most cost effective system was found to be intermittent sand filtration. After passing through the sand filters, the treated wastewater will be discharged to the river by gravity outfall sewer.

By discharging the effluent from Sterling's wastewater treatment plant into the South Platte River during the summer irrigation months, this plan maintains the traditional use of the effluent by downstream water users. The cost of protecting downstream water users from injury was a significant factor in the unfavorable economic impacts associated with alternatives involving new consumptive use of the summer effluent.

Although the waters of the South Platte River are fully appropriated, there are certainly examples of attempts to identify and put to use those waters that may otherwise go to waste, such as winter and storm related flows. Numerous storage reservoirs dot the South Platte River valley which store winter flow. Recently a ground water recharge project sponsored by several interested agencies successfully utilized the diversion of such flows through an abandoned irrigation ditch and natural sinks for augmentation. The winter ground water recharge portion of Sterling's wastewater disposal and reuse plan, in a similar manner, saves that water otherwise lost downstream to Nebraska.

Effluent from the treatment lagoons will be pumped for six winter months to a deep, 40-acre natural sink located approximately one mile south of the lagoon site. The wastewater will be discharged to the surface to infiltrate and percolate through the sandy soils and travel via the unconfined aquifer back to the river. The process of water returning to the river is termed accretion. It was determined that the rate of water returning to the river from any winter recharge program peaks during the following midsummer. Total water returned to the river during the summer months will supplement the flow of the South Platte River, resulting in a reuse plan that potentially benefits rather than injures downstream water users. An integral part of this plan is for the City to claim and receive water rights credits for augmentation of future municipal water needs as a result of the recharge program. This new water right carries a value of $2 million. The right to store winter water through recharge must be granted by the Water Courts. Such rights necessarily will be junior to the rights of the downstream Julesburg Reservoir. In severe drought years, when there is not "free water" in the river, the lagoon effluent can be rerouted through the sand filters and discharged to the river during periods of call by the Julesburg Reservoir. From historic records, it is unlikely that such calls would be made in more than one out of ten years.

The recharge site itself is a natural sink over 30 feet deep with uniform, gently sloping sides. The entire sink is covered with natural grasses and brush. This vegetation will remain as undisturbed as possible during construction of the transmission lines from the treatment lagoons. No need for earth work or diking was identified. TSS in the wastewater used for recharge will be low during the winter and the six-month resting period should eliminate the potential problems of soil clogging. Registered water wells are known to exist in the probable path of greatest ground water flow from the recharge site. For observation, wells will be installed for auditing ground water quality parameters and ground water movement.

CONCLUSIONS

1. Beyond the traditional concepts of reusing treated wastewater for crop irrigation, ground water recharge to develop new water rights, and the development of wildlife wetlands, are reuse opportunities which are worthy of investigation in the semi-arid West.

2. The importance of water rights and associated high costs had an unfavorable impact on alternatives with high consumptive use, such as crop irrigation. On the other hand, alternatives which incorporated ground water recharge and actually provided new water rights, gained a distinct cost advantage.

3. For Sterling, Colorado, the most cost intensive reuse alternative was crop irrigation with winter storage. Energy and labor usage was also highest for those options which incorporated land application for crop production.

4. The selected reuse plan (summer discharge to the South Platte River with winter ground water recharge) had the lowest cost to the City, did not injure downstream water rights, and provided the City a means of developing a new domestic water source.

BIBLIOGRAPHY

Diagnosis and Improvement of Saline and Alkali Soils. US Department of Agriculture Handbook 60. US Salinity Laboratory Staff. Washington, DC. (1954).

Duke, Harold R.; Smitka, Darryl E.; & Heerman, Dale F. Groundwater Contamination from Fertilizer Nitrogen. Preprint 3003. Presented at ASCE Fall Convention and Exhibit. San Francisco, CA. (October 1977).

Duke, Harold. Agricultural Engineer, Fort Collins, CO. Personal Communication.

Heerman, D. F.; Haise, H. R.; & Mickelsen, R. H. Scheduling Center Pivot Irrigation Systems for Corn Production in Eastern Colorado. Transactions of the ASAE. 19:2 (1976).

Heerman, Dale F., Dr. Agricultural Engineer, Fort Collins, Colorado. Personal Communication.

Huff, R. Theodore; Schneider, Paul A. Jr.; & others. Hydrogeologic Characteristics of the Valley-Fill Aquifer in the Sterling Reach of the South Platte River Valley, Colorado. Open File Report. US Department of the Interior Geologic Survey. Lakewood, Colorado (1972).

Irrigation Water Requirements. Technical Release No. 21. US Department of Agriculture, Soil Conservation Service, Engineering Division. (Apr. 1967. Rev. Sep. 1970).

Mass, E. V. & Hoffman, G. J. Crop Salt Tolerance - Current Assessment. Jour. ASCE. 103:IR2 (Jun. 1977).

Process Design Manual for Land Treatment of Municipal Wastewater. US EPA. (Oct. 1977).

Ramey, Victor E. Jr., President & Barden, Ivan, Secretary. Bravo Ditch Company. Personal Communication. (31 May 1978).

Reed, James. Logan County Colorado State University Extension Agent. Personal Communication.

Rhodes, J. D. & Merrill, S. D. Assessing the Suitability of Water for Irrigation. Theoretical and Empirical Approaches. Prognosis of Salinity and Alkalinity. FAO Soils Bulletin 31:69-109.

Rhodes, J.D. Potential for Using Saline Agricultural Drainage Waters for Irrigation. Proceedings of Water Management for Irrigation and Drainage. ASCE, Reno, NV. (Jul. 1977).

Rhodes, James, Dr. US Salinity Laboratory, Riverside, CA. Personal Communication.

Sawyer, C. N. & McCarty, P. L. Chemistry for Sanitary Engineers. McGraw-Hill Book Co. (1967).

Smika, D. E.; Heermann, D. F.; Duke, H. R.; & Bathchelder, A. B. Nitrate-N Percolation Through Irrigated Sandy Soil as Affected by Water Management. Agronomy Journal. Volume 69. (Jul.-Aug. 1977).

Sprinkler Irrigation. Textbook Re-editing Committee of the Sprinkler Irrigation Association. Silver Spring, MD. (1975).

Water Quality Criteria. WPCF. Washington, DC. (Apr. 1968).

TABLE I

WASTEWATER FLOW PROJECTIONS

Projected Average Daily Wastewater Flows

Year	(MGD)	(cfs)	(ac-ft/day)	Yearly Flow (ac-ft/yr)
1980	2.36	3.65	7.24	2,640
1985	2.64	4.08	8.10	2,960
1990	3.00	4.64	9.10	3,360
1995	3.39	5.25	10.40	3,800
2000	3.88	6.00	11.90	4,340

TABLE 2

CITY OF STERLING
REUSE ALTERNATIVES
COST SUMMARY

Parentheses indicate a negative cost or income.

No.	Alternative Description	Total Capital Cost ($)	Average Annual O/M Cost ($/yr)	Equivalent Annual Cost ($/yr)	Average Annual Cost To City ($/yr)
1	Year Round Discharge to South Platte River	1,013,000	23,500	116,700	52,000
2	Summer Discharge to South Platte River with Winter Recharge	1,465,500	36,250	166,500	(38,700)
3	Summer Discharge to South Platte River with Winter Wildlife Enhancement	1,899,000	38,900	203,100	130,200
4	Summer Discharge to Bravo Ditch with Winter Recharge	1,606,000	36,250	177,600	(26,300)
5	Summer Discharge to Bravo Ditch with Winter Wildlife Enhancement	2,039,500	38,900	214,100	142,500
6	Summer Discharge to Bravo Ditch with Winter Discharge to South Platte River	1,153,500	23,500	127,100	54,000
7	Irrigation of Iliff Interchange Site with Winter Storage	8,444,900	215,050 (242,250)[1]	598,400	553,400

TABLE 2 (Continued)

CITY OF STERLING
REUSE ALTERNATIVES
COST SUMMARY

No.	Alternative Description	Total Capital Cost ($)	Average Annual O/M Cost ($/yr)	Equivalent Annual Cost ($/yr)	Average Annual Cost To City ($/yr)
8	Irrigation of Iliff Interchange Site with Winter Recharge	6,376,900	142,100 (144,100)[1]	464,200	285,300
9	Irrigation of Iliff Interchange Site with Winter Wildlife Enhancement	7,186,400	145,000 (144,100)[1]	535,800	487,700
10	Irrigation of Iliff Interchange Site with Winter Discharge to South Platte River	6,482,900	136,000 (144,100)[1]	471,500	410,000
11	Irrigation of I-76/RR Site with Winter Storage	7,073,900	232,250 (215,150)[1]	543,600	553,400
12	Irrigation of I-76/RR Site with Winter Recharge	5,845,400	142,500 (128,300)[1]	445,200	271,200
13	Irrigation of I-76/RR Site with Winter Wildlife Enhancement	6,782,900	146,100 (128,300)[1]	527,500	481,800
14	Irrigation of I-76/RR Site with Winter Discharge to South Platte River	6,079,400	137,100 (128,300)[1]	463,200	405,800
15	Irrigation of Sonnenberg Site with Winter Storage	8,510,100	246,200 (204,000)[1]	675,100	683,900
16	Sale of Summer Effluent to Bravo Ditch	2,999,700	28,600 (16,400)[2]	238,700	54,000 (154,000)[3]
17	Storage and Sale of Summer Effluent Discharges	4,967,000	33,600 (15,500)[2]	391,400	54,000 (301,200)[3]
18	Storage and Sale of Year Round Effluent Discharges	5,733,900	89,200 (19,200)[2]	519,500	54,000 (510,200)[3]

[1] Revenues from crop sales and wastewater fertilizer credits.

[2] Wastewater fertilizer credit. [3] Cost per year to irrigator.

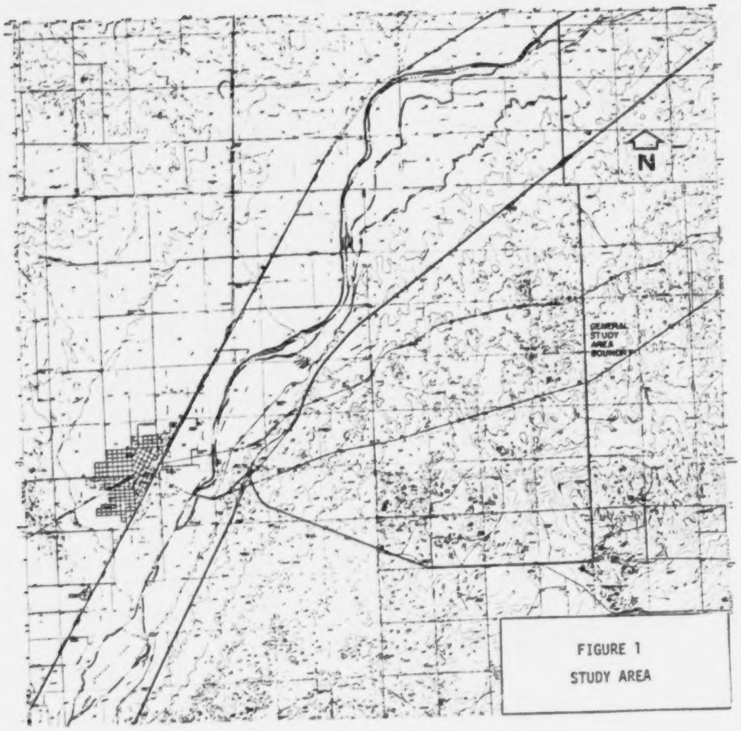


FIGURE 1
STUDY AREA

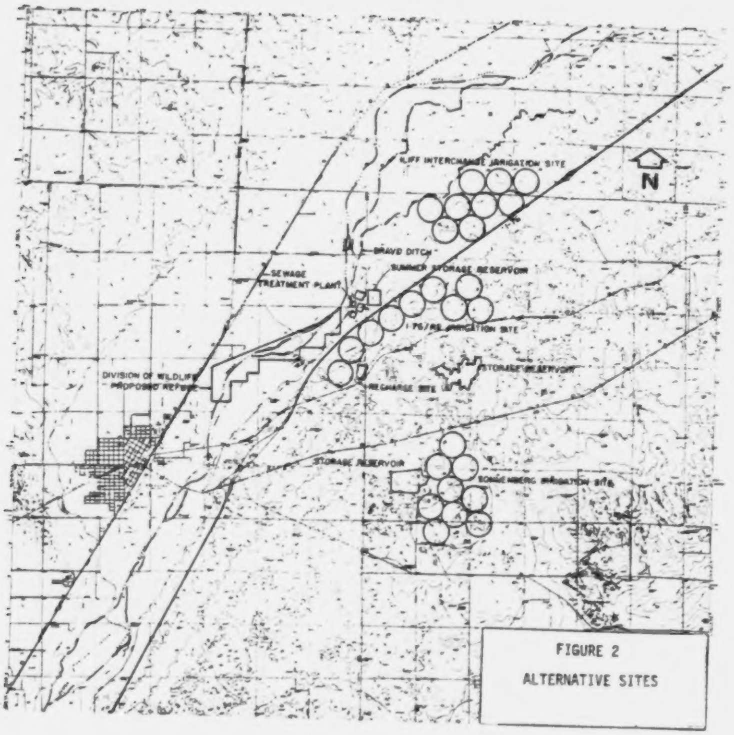
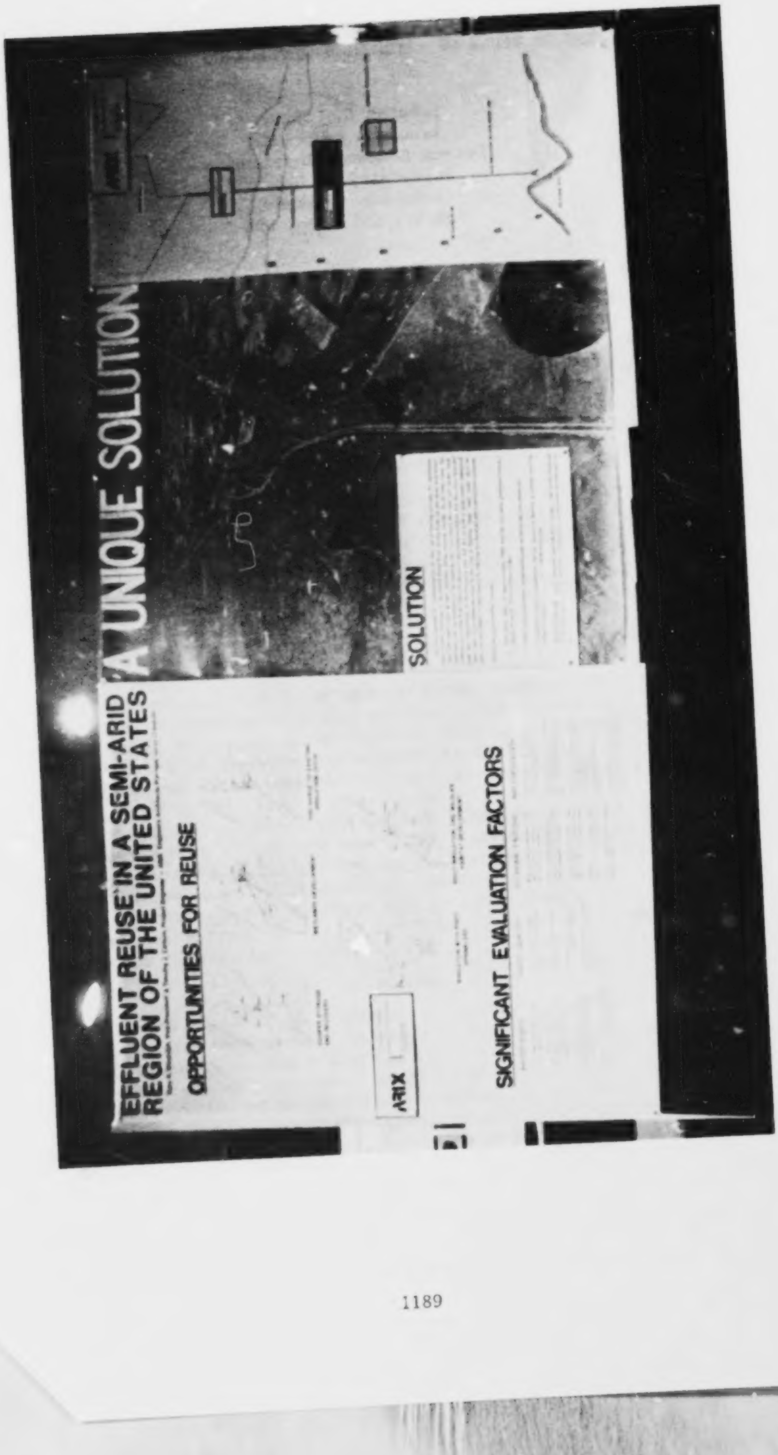


FIGURE 2
ALTERNATIVE SITES



EFFLUENT REUSE IN A SEMI-ARID REGION OF THE UNITED STATES

OPPORTUNITIES FOR REUSE

SIGNIFICANT EVALUATION FACTORS

A UNIQUE SOLUTION

SOLUTION

A PRELIMINARY ANALYSIS OF THE POLITICAL TENSIONS AFFECTING PLANNED REUSE OF WATER IN THE COLORADO RIVER BASIN

Loretta C. Lohman
Research Associate
Denver Research Institute
University of Denver
(Colorado Seminary)
Denver, Colorado, USA

Unplanned reuse is a regular occurrence as the Colorado River flows over 1,400 miles of weathering soils and rocks before it enters Mexico. Increasing salinity, both from natural and man-created causes, as well as inadequate supplies to meet the demands of a developing region are also factors in managing the Colorado River. Given these conditions it is likely that planned reuse of water in the Colorado River Basin might not only increase beneficial utilization of existing supplies, but might also improve water quality if known supplies of low quality water are tapped for consumptive reuse.

The technological and economic problems of the reuse of low quality water are the subject of much research and are fairly well identified. Legal barriers are identifiable, and are also the subject of specific research. Institutional barriers to planned reuse, as affected by the political processes supporting them have, however, been scarcely examined.

Introduction

Since the most commonly accepted meaning of the term institution is that of a corporate body--a university, a government, a particular agency--a careful definition of institution as a less tangible but nevertheless clearly distinguishable process must be drawn in an attempt to separate the law from its administrative implementation.

This paper defines an *institution* as the actual operating activities, as shaped by administrative policy and precedent, of a person or agency charged with application of legally defined water management processes. This is a narrowed, politically oriented definition, establishing a direct focus on the agencies and actors most concerned with the activities involved in the allocation and uses of water--and, consequently, on the potential (and actual) tensions which affect planned reuse.

That such a definitive focus is necessary to identify and clarify complex processes, the author's research experience attests. For example, many interviewers, both officials and water users, when asked the relatively straightforward question, "Do you encounter or can you identify any institutional barrier to water reuse--that is, the accepted practice, customary attitude, or non-mandatory policy of a water management entity?," responded, "None." Nevertheless, further inquiry focusing on administrative implementation would elicit mention of bureaucratic delays, of incomplete information concerning rules and regulations, or of sentiments that reuse was already occurring or was of little value. Occasionally a respondent would mention a particular law or method of enforcing the law as a barrier under the above paradigmatic description of institutional.

Concomitantly, interviewees frequently complained that "so and so in the other office doesn't understand the law," or, "EPA is living in an unreal world." Only continued questioning and repeated explanation of the meaning of institution produced discussion of those areas in which attitudes toward reuse and toward administrative flexibility may influence reuse proposals.

For this reason it is important to note that the relatively new area of public administration analysis, encompassing our definition of institution, has not developed the documentary base or sophisticated tools which are necessary to deal with the complex and political nature of water administration in the Colorado River Basin. Thus, the following discussion should be recognized as a preliminary description of those elements which merge to form the management--the institutional entities--of the Colorado River. This description should provide the foundation for development of a more sophisticated set of analytical tools, including creation of an in-depth questionnaire and several opportunities for thorough case studies which would provide a data source for more comprehensive understanding of the processes at work.

It is not the intention of the author to present a full factual detailing of the institutional processes within the Colorado River Basin. Because this is a relatively unstudied area with little written documentation, the material presented is based on three years of intense observation combined with a series of specific questions addressed to those involved in the institutional processes. Since the intent of this paper is not to offend, but to illumine, all but the most public names have been omitted and the conclusions have been generalized. Thus, although one agency or one actor may not precisely fit into this paper's description, the general pattern is established. It is also fair to note that these patterns and individual actors are, in the end, governed by the law and any activities which are determined to be arbitrary, capricious or discriminatory are answerable under that law.

However, it is reasonable to state that, while legal responsibilities and obligations provide the framework in which agency operations take place, each agency, in addition to its specific legal character, is part of a larger governmental entity with broader and even different purposes and goals. These factors influence that agency's approach to the direct responsibilities at hand. In addition, the individual personalities who both shape and are shaped by the agency bring all the prejudices and experiences that form their perceptions to bear on those activities and reflect the tensions inherent in the management of the waters of the Colorado River Basin.

An agency, then, can from its inception contain contrary impulses which can propel it into conflict with other agencies, both federal and state. In addition to this potentially volatile character, the interplay of various actor's personalities sets the stage for political intrigue. The Colorado River Basin is a particularly apt theater for this drama, because it provides a classic example of the various elements that can combine in an administrative action. The Colorado is an international stream, of concern to United States foreign policy goals. It is a regional river, in a water-short area, and controlled in part by interstate agreements. Finally, it is a river that is subject to seven sets of state laws and policies. Within this complex managerial system the Colorado River is also pressured by problems of rising salinity coupled with growing intense demand on the quantity of water available for competing uses.[2] Thus the elements which impact on a single management decision take a pyramidal form, moving from broad federal interest through regional concentration and state priorities to a peak in agency personnel perceptions.

National Interests

Because the last forty miles of the Colorado River form the border between the Mexican states of Sonora and Baja, California, it has become a matter of international and, therefore, national interest. In 1944 the U.S.A. negotiated with Mexico the quantity of water to be delivered to that country, and in 1972 the quality (specifically salinity) of that delivery. In both instances the Colorado River was a foil for negotiators intent on achieving a broader goal than the narrow issue of water deliveries. Since the river plays a role in the maintenance of international comity, Congress, in 1968 and in the 1973 salinity control legislation, declared delivery of specific amounts of water meeting the agreed upon quality standards to be a national obligation.[1]

The ability of the Colorado River Basin states to coalesce for their mutual benefit led to negotiations with the federal government which resulted in the salinity problem becoming a national obligation. Thus the structural emphasis on solutions to rising salinity is overseen by the U.S. Bureau of Reclamation and monitored by a consortium of the seven states and other federal agencies such as the U.S. Environmental Protection Agency.

However, as is often the case, national interests in the years following passage of the Salinity Control Act have changed and expanded. Broad environmental concerns, many initiated by national citizens' coalitions, have become law. These new interests affect not only the structural aspects of the salinity control program but also create conflicts as to the actual uses of water. Peripheral environmental concerns, such as air quality, have impeded projects which would reuse and consume low quality water. General wildlife protection, threatened and endangered species lists, historic preservation, and a broad range of pollutants other than salinity are all factors which impact on salinity control and water reuse projects.

All of these areas of national interest, joined with a new national water policy which emphasizes conservation and non-structural remedies to water shortages, create an atmosphere of conflicting regulations and demands. The focus of federal regulations aims at causing state and local entities to take specific actions in order to receive desired funding or needed approvals. Under broad national rules conflicts arise concerning the reasonableness or practicality, not to mention the legality, of a specific action in a particular locale.

The complex managerial demands placed on the Colorado River Basin extend from the location of a water use to the sophisticated demands of international policy. However, all levels of government exhibit similar patterns of agency formation, clientele relationships and agenda building. Thus, this report will address the more specific aspects of the actors' effect on administration in its general appraisal of agency/administrator roles.

Regional Forces

The recent history of water management in the Colorado River Basin is an example of *distributive politics* (i.e., a form of decision-making wherein a coalition of multiple sets of actors bargain to reach a consensus position out of differing, and sometimes conflicting, interests). Basin-wide coalitions have formed to bring about compromise proposals for desired actions in water development and salinity control projects at little or no cost to states or individuals. The Colorado River Basin Salinity Control Forum is a coalition at work to obtain the desired goal of salinity reduction as a national, rather than a state or regional, responsibility. The Forum was created by the seven Basin states in response to several questions generated by passage of PL 92-500, not the least of which is the dilemma of easing potential burdens of salinity control into federal hands while protecting each state's right to develop its water resources.[3] Helen Ingram has described the process of distributive politics which influenced passage of the 1968 Colorado River Basin Act as a classic example of that process untainted by the various new pressures that are now found in the Colorado River Basin.[4]

The Ingram case study describes the traditional pattern of distributive--disparagingly termed "pork barrel"--politics. However, an unfamiliar element of *regulatory politics* has been injected into the system--that is the conflicting contemporary demands of quality goals versus development pressures versus traditional patterns of water use and water management. As has been noted, federal and state environmental interests coupled with a national water policy which emphasizes conservation at the expense of structural water project development have created tensions between various levels of government. In addition, these same factors could tend to isolate the seven Basin states from each other, and could further complicate the intra-governmental relationships between water allocators and those charged with protecting water quality.

Interstate cooperation, however, is aided by the consensus-type format exercised by the Colorado River Basin Salinity Control Forum, as well as by the seven states' implicit agreement as to the course of river development which has existed, with fairly strong continuity, since the legislation which created Hoover Dam was enacted. Also, the governors of the seven states have (except for New Mexico's single appointment of an allocator who is also a member of the Water Quality Control Commission) seen fit to appoint both a water allocator and a water quality administrator as Forum representatives. In consensus proceedings most of the interstate policy differences are compromised in order to present a single view to the federal agencies which receive Forum input. Meeting the broad range of issues impacted by salinity control activities remains a collective Colorado River Basin effort as far as federal measures and regulations are concerned,[5] most actively under the auspices of the Forum and its partner body which acts as an advisory council to federal water agencies. Other regional organizations serve as forums for water-related issues, but the membership or staffing is essentially comprised of the same state experts who make up the Forum.

The informal, as well as formal, ties joining the water experts of the seven states do not always carry over to intrastate activities. Nor does a regional policy on salinity control issues necessarily extend to consensus on the broad range of development and environmental issues confronting water managers in the Colorado River Basin. Varying state policies as well as differing agency agendas can impede a smooth process of water management and can become institutional barriers to specific aspects of development, such as planned non-potable reuse of water. The area where the conflicts inherent in regulatory politics breach the consensus aspects of distributive politics forms the crux of an institutional analysis of the water management process in the Colorado River Basin.

State Policies

The individual actors who perform the administrative duties which can impede or facilitate a particular project are subject to one other outside force in addition to those previously described: the popularly elected state governor/legislature generally has a policy which impacts on the management of the state's water resources.

Such state policies are particularly critical in the Colorado River Basin for three major reasons: its economy, with the exception of Nevada's portion, is traditionally agrarian; out-of-basin exports supply water for millions of people; and finally, the Upper Basin contains rich energy resources the development of which impacts on the environment and the economy as well as on the traditional water-use pattern. These factors mesh with the already serious problems of water supply and salinity control to form the basic information upon which any policy must be based.

Although the policymakers in the seven states have come together to obtain passage of major structural developments affecting the Colorado River Basin, including salinity control projects, they have not presented a coordinated regional plan for the actual use of water. For example, the Governor of Colorado, Richard Lamm, has made the preservation of irrigated agriculture a very high water priority,[6] while the former Governor of Utah, Calvin Rampton, was more interested in energy-industrial development.[7] Policies in the remaining states cover a wide spectrum of the problems of development and water use. In terms of institutional analysis, what such policies state is less important than the fact that they exist and act as an agenda controlling force in the process of resource administration. A particular emphasis or direction, such as California's encouragement of reuse, in which water management is a factor, influences administrative proceedings as well as the activities of current and potential water users.

Noteworthy, in this respect, is not the type of control but the methods of exercising authority over water-related matters with which state policymakers attempt to achieve control. Governor Matheson of Utah, for example, has expressed a desire to change basic water allocation methods to include a beneficial use formula that would "more directly represent the public interest."[8] Since policymakers generally define the public interest, this change would expand their (in this case Matheson's) role in defending primary state control over the management of water resources within state boundaries to include participation in the management process. Colorado, as have other states, has found an alternative means to the ends of preserving state control and forwarding state policy by signing an agreement with the Department of Interior which grants the state the right to approve or disapprove the allocation of water from federal water projects to energy industry uses.[9] The prerogative granted by

Interior assists Colorado policymakers in maintaining the historic use of water in agriculture.

An additional vehicle available to policymakers is the rousing of public support by the traditional means of campaign-style rhetoric and charismatic politicking. For example, the export of water from a state by its use as the liquid medium for coal slurry can provide opportunity for this emotional approach to policymaking. Historic prejudice against the loss of any water imposes a policy of extreme conservatism toward the export of even universally recognized unusable water, and can, if the policymaker opposes export, provide the theater for an exercise in political pyrotechnics. More conventional participatory politics have resulted in each of the states of the Colorado River Basin exercising control over the export of water (as is described in the final section of this paper), although none of the restrictions is insurmountable. Because water is an emotional issue in the region, political expedience requires that even administratively permissible export both achieve legislative approval and identifies major secondary benefits such as the improvement of water quality.[10]

Finally, development of each state's full compact allocation of Colorado River water (which requires the full panoply of consensus politics) in ways consistent with each state's economic and growth policies is a major concern of policymakers. This can in itself have a complex double edge in that reaching full development threatens the interests of other states and puts severe strains on the coalition which serves as the means to accomplish development. Although inter-Basin cooperation is currently stable, it enjoys a relatively short history, and the successful exercise of distributive politics is being pressured by both regulatory politics and specific developmental interests. This is creating renewed support for the cooperative endeavors of distributive politics and for continuing efforts for compact preservation. Witness a Colorado gubernatorial candidate's recent statement of intent to file suite concerning the Colorado River Compact in order to clarify what is the state's share of water, and the subsequent reaction of Basin policymakers. They believe such a suit will jeopardize regional cooperation in seeking state development, and might even start a "civil war" between western states,[11] since the Colorado River Compact has become a concept which forges regional cooperation in protecting states' rights, as well as in promoting federally financed programs. This concept of the Compact, however, could in itself inhibit innovative reuse programs which might impact on obligations since discussion concerning Compact construction is not an acceptable agenda item in public forums.

Role of Agency/Administrator

All of the previously discussed factors come to bear on the actors who are on the line, the people who make up the water management agencies and who take the daily actions necessary to administer the varied elements of a water resource. In most of the Colorado River Basin states these actors fall into two fairly identifiable agency-clientele categories: water allocator interests and water quality interests. Even in California, where both interests are housed in one department, the outside influence of a health agency serves as a locus for water quality clientele. Similar patterns of agency-clientele relationships are found in the assignment of responsibilities to federal agencies.

The terms water allocator and water quality interests are used here as general labels and do not prevent recognition of the full range of activities required of a person or agency so designated. For example, an allocator may be involved in conservation activities, waste discharge permits, and he may--as in four states (Arizona, Colorado, Nevada and New Mexico)--be a member of the designated water quality overseeing board. However, recognition of the full roles of the allocator and the water quality actor does not preclude an examination of their impacts on water reuse.

Although the theoretical solution to the conflicts inherent in agency separation would be formation of a single umbrella encompassing each state's water resource management, there are several reasons why responsibility has been divided. One reason is the relative newness of water quality issues as a matter of public concern. The attitudes of policymakers toward water quality issues often conflict with long established water allocation interests, thus making new agency creation a more politically astute short-term solution. Finally, it is simpler to create a new agency than to restructure an existing one to encompass a range of incompatible goals.

The potential conflicts inherent in the differing agency-clientele goals are illustrated by the kinds of responsibilities and administrative actions that each agency must undertake. An allocation agency attempts to achieve the maximum beneficial utilization from a water resource, while a quality agency attempts to maintain a given standard of quality even if that goal precludes full utilization. The fact that water allocation agencies and interests have a long and well developed legal and administrative history in the Colorado River Basin has tended to create an elite among water managers. The actors who people these agencies most often share a common professional history and a mastery of a difficult and arcane technical subject affecting everyone, as well as similar training and occupational biases. Agency leadership, although often subject to political control, tends to follow a generational pattern and to form a strong sense of tradition. Allocation agency-clientele respond in a similar traditional manner in which the market system is the primary source for change. Conversely, consideration of water quality is a relatively new concern in which research as to what is a pollutant and what is the desired quality is still ongoing. Without a solid empirical foundation of information, these agencies are required by law to protect and enhance water quality according to a set of still changing standards. Water quality agency personnel, because of a newly developing and unsophisticated skill pool, are frequently drawn from agency clientele, which can create a hardening of positions and set up conflict with the more entrenched allocators.

While the differing roles of the agencies do not, in themselves, create an institutional barrier to reuse, their adversary positions can impede approval on any given project through delaying requirements prescribed by either agency. At the worst, open warfare can erupt between the two agencies, which can lead to extreme interpretations of regulations and slow normal management processes. The differing organizational structures of water resource agencies establish an adversarial climate and the differing clientele reinforce that atmosphere. It is nevertheless the primary actors, as they perceive their roles, who shape legal obligations into a particular administrative process. This role identification performs a dual function: it guides the development of policy which guides administration of legally defined duties, and it defines the type of relationships that are established between the agencies.

As previously noted, water allocators, because of historical and circumstantial factors, have become a type of elite and their perceptions are influenced by that position. The role played by an allocator in addressing his legal duties to obtain beneficial use, to prevent waste, and to protect the rights of prior appropriators can critically affect the potential for water reuse. For example, since the Upper Basin hasn't reached full use of its Compact allocations, an allocator in one of these states can exercise a certain amount of flexibility in handling new requests for appropriation. Within legal and judicial constraints, he can allocate the amount of fresh water requested if that amount is available. Or, he can actively promote reuse by severely restricting the amount of water requested through re-definition of beneficial use, by a restrictive factoring of historic consumption, or by involving the concept of public interest (even if the concept is not legally recognized) and he can work with other state agencies to encourage the use of reclaimed rather than fresh water. In states with more limited fresh water resources, an activist allocator can, in addition to applying the aforementioned concepts, strictly interpret regulations concerning a change in use or change in diversion point. Such an allocator might also bring the power of his office to bear on the myriad negotiations which might be needed to satisfy all parties affected by a reuse project.

On the other hand, an allocator can be passive, performing his legal obligations in a traditional manner without making any effort to either impede or assist a proposed reuse project. While he might know, and even state, that a particular proposal is flawed because of negative impacts on other users or on the historic regimen of the river, he will not initiate efforts to stop the project or take steps to solve the problem, but will wait until those who are injured take their own action. Although he has performed all administrative functions, his net effect is to hamper reuse proposals because of the uncertainties he could, at least in part, resolve.

Because the allocator is in an established position, he can, if he chooses, remain relatively untouched by the demands put forth by water quality interests. Certain quality regulations might affect his decisions, but for the most part his authority is unchallenged. However, the Basin-wide forces of distributive politics have required some allocator response to the quality problems of salinity control. Once regulatory politics (salinity control issues) have entered this static arena, it opens the door to the importuning of other non-traditional interests. The interface between the allocator and the quality administrator becomes the critical factor in determining the administrative attitude of each agency. If no common ground is sought or developed, the administrative processes of both agencies, although most drastically those of water quality, can be adversely affected.

The strain between water management agencies derives, in some significant degree, from the fact that allocators usually hold the dominant role both legally and politically. Furthermore, the strong agency-clientele relationships found in quality agencies and the role perceptions of quality administrators drawn from an often ideological clientele and from public-health oriented agencies can contribute to inter- and intra-agency tensions. In this light, the operating thesis of mission-oriented (quality) agencies is too often based on immutable goals that preclude necessary flexibility in dealing with allocators. More elastic--and mutable--concepts, such as "reasonable degradation" and "waste-load allocation," as well as consideration of the quality of a stream in comparison to the quality of a discharge, could provide a focus for compromise.

When the opposing roles of the two agencies are respected and recognized by each other as having some value, water quality efforts can become the most direct facilitators of reuse. Not enjoying the more discreet interpretive tools open to an allocator, quality administrators face a clear obligation to prohibit or control direct return to a stream system. Thus, discharge prohibitions, especially when coupled with requirements for non-permeable evaporation ponds, can make reuse economically attractive. However, if inter-agency communication is to be kept open, the quality administrator should incorporate provisions which allow discharge of usable water which does not impair stream quality, thus freeing that water for continued beneficial use. Since data are not complete on what exactly constitutes a pollutant, these actors have a further burden in setting standards in light of demands on the water supply and its traditional uses. In this sense, standards designed with the assumption that deliberate reuse of water is bad, as is the case with certain health safety requirements, can be the tool for enforcing an institutional barrier.

Water resource agencies in several Colorado River Basin states have begun to discover opportunities for communication, partly in response to what each agency perceives as unreasonable federal regulatory pressures, and partly in recognition of the fact that quantity-quality concerns must be meshed to meet the greater problems of resource management. Although acknowledging persistent differences in methods used and attitudes of quality and quantity interests, both California and Nevada have provided a common goal in creating state policy promoting the desirability of reuse. Mutual interest in fully developing compact allocations, coupled with an Upper Basin lack of severe pollution problems and surplus of fresh water, have created areas of agency compatibility in other states. In one state the allocator, exploiting his tenure, elite position, and membership on the commission directing water quality control policy, has managed to create an atmosphere of authority which has allowed him, in essence, to become the quality agency's mentor. An uneasy climate still exists in two states where strong role identification and/or agency-clientele relationships exist for the actors. This last situation could easily recur in each state as the divisive impact of regulatory politics makes increasing demands on water resources.

The administrators in each of the seven states, within legally defined limits, have a degree of administrative flexibility which can be a passive or an active management resource in achieving desired goals. An administrative attitude that considers water reuse to be of minimal value except for some water quality benefits forces the management process into a passive and negative direction, away from potential reuses. On the other hand, an administrative attitude that believes reuse can be a positive contributor to solving both water quality and some water supply problems can turn an agency toward active pursuit of reuse opportunities. In the end, shaped by the various forces which may mold him, the individual actor and his particular mind set determine whether or not an institutional barrier is erected. If the actor has himself become an "institution" in terms of longevity and the exercise of power, his ability to create or remove barriers is enhanced. If the actor has yet fully to command his milieu, a barrier may inadvertently emerge from the confusion. In either case, the actor's role perception combined with all the influencing factors described in this paper finally determines whether or not an institutional barrier exists and whether or not it can be overcome.

Summation of Thesis

This paper has traced the origins and evaluation of the varied elements which make up the complex institutions responsible for water resource management in the Colorado River Basin. From this analysis of the complexities of these institutions as they impact on planned water reuse, several themes have emerged and are worthy of summation.

First, the task of defining and describing the institutions themselves is an exercise in complexity. Every level of government, as well as several kinds of social pressure, impacts upon the action strata we have defined as a water resource institution. Since the issue itself is so complex, no simple solutions exist, either for analysis purposes or for practical goals.

Second, the demands of regulatory politics--all the pressures for salinity control, for environmental protection, for energy development--have disrupted the functioning of the process of distributive politics. To some degree, these forces of conflict have negated the work of coalition politics by unsettling interstate and intrastate communication. Again, analysis discovers no simple solutions for bringing widely contradictory interests into an arena of compromise.

Third, in the end, it is the actors and their activities that are at the heart of the entire issue. They are the institutions. They are the barriers or the facilitators. They are the focus of all the complex and contradictory forces which have a stake in water resources.

Politics in Practice

The case studies the author conducted as part of her broader research into barriers to water reuse in the Colorado River Basin have indicated that legal and economic problems were foremost in most current reuse projects. However, an as yet unenacted proposal to take low quality springs water out of the appropriable river system and divert it for use as the transport medium of a coal slurry pipeline does illustrate some of the political problems any large-scale reuse project might face. Taken in summary form, this case study demonstrates the thesis that river management is a complex issue containing contradictory impulses and is dependent upon the actors involved.

Initially, in this instance, a substantial quantity of saline water (11.5 cfs at 9,650 mg/l total dissolved solids), issuing from a natural springs, was selected by the U.S. Bureau of Reclamation for a point-source salinity control project involving diversion, desalinization and make-up water of about 2,740 acre-ft. Since this is an extremely expensive proposal with a relatively minor impact on overall

Colorado River salinity, there has been a search for alternatives to desalinization, foremost of which is the use of springs water to transport coal slurry.

The area's responsible authority, the state water allocator, has declared coal slurry pipelines to be a beneficial use of water, thus allowing a more thorough investigation of project feasibility. However, he is not sure if the removal of 11.5 cfs can be replaced by unused area waters and he may be forced to prohibit the use of the saline waters on the grounds of damage to other water rights holders.

The U.S.A. agency which is planning the desalinization project would prefer not to undertake the expensive project but must, under law, remove the springs because it is a contributor of saline water. The federal agency also must protect the area habitat of a small fish--which has been placed on the list of threatened and endangered species--and therefore may not allow any activity which would disrupt the habitat.

Water quality interests in the state support the use of the springs for coal slurry wholeheartedly, pointing out the beneficial use, the savings in not desalting the water, and the potential benefits to the river system. They are not particularly concerned with the technical problems associated with construction of a pipeline, and feel that damage to other users would not present a major obstacle.

Area water users, on the other hand, are generally opposed to any activity on the springs. However, they much prefer a certain supply following desalinization to the consumption of a coal slurry pipeline. The area exhibits the strong anti-export of water feelings prevalent throughout the western U.S.A. Such feelings are transmitted to and reflected by elected members to the state legislative body who exercise some control over major water issues.

Finally, the power company interested in building the pipeline has conducted a feasibility study on the use of the springs as a water source. Although the company had essentially predetermined that the use of the springs presented prohibitively expensive geographical barriers, the study was conducted as part of the political price necessary to obtain sufficient water. Needless to say, the feasibility study found that use of the springs would be so expensive that the cost would outweigh the benefits.

All of these elements have converged over the use of one small saline springs. International, and therefore national, interests require that the salty water be removed from the river system, although national interests at the same time require protection for an obscure fish. Similar conflicts affect the water allocator who has a duty to protect area water users and to promote beneficial use of water supplies. Water quality interests, local water users, and the power company all have positions unhampered by the subtleties of law and differing clientele--they are either for or against the use of the springs in a coal slurry pipeline based on their perceptions and their needs.

In many instances the impetus to reuse is stronger--either politically, economically, or sheer necessity--than any barriers erected. It still remains to be seen if the incentives to use the springs water are strong enough to overcome a most severe set of political and physical barriers.

ACKNOWLEDGEMENTS

This paper is in part derived from two University of Denver Research Institute studies, one funded by the U.S. Environmental Protection Agency, Region VIII, under Contract No. 68-01-3578, and another funded by the U.S. Department of the Interior, Office of Water Research and Technology, under Agreement No. 14-34-0001-7816. Professor Terence J. Lohman contributed precise editing and stylistic guidance.

REFERENCES

1. The Colorado River Basin Project Act of 1968, 43 U.S.C. (1970); Pub. L. No. 93-320, 88 Stat. (1974).
2. MANN, DEAN E. Water Policy and Decision-Making in the Colorado River Basin [n.p.]. Lake Powell Research Project Bulletin No. 24. (July 1976). pp. 1-23.
3. COLORADO RIVER BASIN SALINITY CONTROL FORUM. Water Quality Standards for Salinity Including Numeric Criteria and Plan of Implementation for Salinity Control. (June 1975). Appendix D.
4. INGRAM, HELEN M. Patterns of Politics in Water Resource Development: A Case Study of New Mexico's Role in the Colorado River Basin Bill. University of New Mexico, Institute for Social Research and Development, Albuquerque. (Dec. 1969).
5. Interstate comity is illustrated by the COLORADO RIVER BASIN SALINITY CONTROL FORUM. Proposed 1978 Revision, Water Quality Standards for Salinity Including Numeric Criteria and Plan of Implementation for Salinity Control. (August 1978).
6. Rocky Mountain News. Lamm Seeks Veto on Water Usage. (March 13, 1977). p. 5.
7. Denver Post. Diversion of Water for Energy Studied. (December 15, 1976). p. 29.
8. Denver Post. Area Water Control Urgent, Users Told. (December 16, 1977). p. 22.
9. Denver Post. Andrus Signs Pact on State Water Use. (July 13, 1977). p. 3.
10. Denver Post. Herschler Can't Support Coal Slurry Bill. (October 26, 1977). p. 23.
11. Rocky Mountain News. Lamm Hits River Pact Suit Idea. (October 25, 1978). p. 9.

The reader may also wish to consult:

FOSS, PHILIP. *Institutional Arrangements for Effective Water Management in Colorado*. Completion Report No. 88. Colorado State University, Fort Collins (Nov. 1978).

MANN, DEAN E. Politics in the United States and the Salinity Problem of the Colorado River. *Natural Resources Jour.*, 15:1:113 (Jan. 1975).

MANN, DEAN E. Conflict and Coalition: Political Variables Underlying Water Resource Development in the Upper Colorado River Basin. *Natural Resources Jour.*, 15:1:141 (Jan. 1975).

PETERSON, DEAN & CRAWFORD, A. BARRY. *Value and Choices in the Development of the Colorado River Basin*. University of Arizona Press, Tucson (1978).

LEGAL INNOVATIONS TO PROMOTE PLANNED REUSE OF WATER IN THE APPROPRIATION-DOCTRINE STATES OF THE AMERICAN SOUTHWEST

Anthony S. Trumbly
Research Associate
Denver Research Institute
University of Denver
(Colorado Seminary)
Denver, Colorado 80208

Introduction

The water allocation laws of the western states dictate the methods by which the right to reuse municipal wastewater is acquired. Barriers to planned reuse of wastewater may be found in both federal/interstate and intrastate systems governing water allocation. This presentation describes the legal structures affecting reuse and the means of operating within those structures in order to implement or promote planned reuse.

The first section of the presentation concentrates on federal and interstate allocations of water. Such allocations include interstate compacts, federal water development legislation, and the judicial decisions which interpret them. The second section discusses the water allocation processes of the individual states of the Southwest. The potential for planned reuse in each of seven states is analyzed, and schematic representations of two states' water allocation systems are contained in an appendix to the presentation.

Federal and Interstate Allocations of Colorado River Water

Allocations Made by Compact or Treaty. The two major agreements which allocate water resources of the Colorado River are the Colorado River Compact[1] and the Mexican Treaty and Protocol of 1944.[2] Other agreements made on a regional basis are derivative of the Compact, as are Congressional and judicial determinations of proper divisions of the water of the River.

The Colorado River Compact of 1922 allotted the waters of the river between an upper and a lower division of the basin, separated at Lee Ferry, Arizona. Under the terms of the Compact, as adopted by Congress in the Boulder Canyon Project Act,[3] the Lower Basin was assured that depletions in the Upper Basin would not prevent at least 75 million acre-feet (maf) of aggregate flow in each successive ten-year period from reaching the Lower Basin at Lee Ferry. The Lower Basin received a guaranteed ten-year, not annual, minimum flow. Experience since the adoption of the Compact has shown that there is less water available in the river than anticipated. The result is that rather than receiving equal parts of the water of the Colorado River, the upper division is entitled to varying and generally lesser amounts of water.

Under the Mexican Treaty, the United States guarantees the delivery to Mexico of 1.5 million acre-feet per year of water from the Colorado River, regardless of the source.

Under the supplemental obligations for the treaty found in Minute 242 of the International Boundary and Water Commission[4] the United States has undertaken an obligation to maintain the salinity of the Colorado River water passing into Mexico at a differential of 115 ppm (± 30 ppm) between the last American diversion and the delivery point to Mexico.

Effect of Interstate Allocations of Water. The allocation of Colorado River water among the states of the CRB provides an ultimate limit on the consumptive use[5] of river water which can be made in each of the states. This limitation would not necessarily affect the type of consumptive use to which a state's apportionment is devoted, but the states would be individually forced to choose among water-consuming alternatives.

Reuse of water for a particular purpose is just as consumptive as a first use of water for that purpose. Any consumptive use of water will be charged against the obligations of the state as a whole. Since consumptive use is limited, each state will be able to maximize its beneficial use of water only be reallocating among in-state users the consumptive use of certain amounts of water. In the absence of judgments as to the relative value of various uses of water, there is no particular incentive to reuse water from a quantity viewpoint. There are exceptions, of course, where the reuse might be either more physically efficient than a first use of the water (such situations are rare; it is more likely that water would be less efficiently used in a reuse than in a first use because of quality degradation) or might contribute to the overall ability of the state to utilize its apportionment to maximum benefit by consuming lower quality waters. Where a reuse option can consume low-quality waters, uses which require relatively good water can operate more efficiently, thus increasing the total benefit gained from the state's share of Colorado River water. Reuse may also be preferable to a succession of first uses where transmission losses of water would be minimized by use of water in one location.

Federal-State Relations. The recent Supreme Court ruling in California v. United States[6] may have a significant impact on the potential for planned reuse of water distributed under the federal reclamation program. The decision in California requires Bureau of Reclamation compliance with state law in both the appropriation and distribution of water. The appropriate state authority may not only impose conditions on the federal government, but apparently will retain continuing control over some aspects of the federal operation. Changes in the operation of a reclamation project could later be mandated by state law and policy in the same way that private water rights are subject to state control. The prevailing position until the ruling in this case had been that the federal government was obliged to comply with state law only in the acquisition of water rights, and not in the operation of reclamation projects. In 1963, it was established that federal water rights acquired under state law were not subject to state regulation to the same extent as other rights. The only remedy for senior appropriators injured by federal actions was money damages; restriction of federal rights was not allowed.[7] Section 8 of the Reclamation Act was considered to be a financial doctrine only--the only federal conformity with state law being in the determination of the property interests for which federal compensation was required.[8]

The effects on planned reuse of recent developments in federal-state relations will stem from loss of federal control over large amounts of water in the CRB. Planned reuse has been favored by the federal government, particularly where water quality benefits result from the reuse. While several of the states in the CRB have shown a statutory or regulatory preference for planned reuse, others remain neutral toward the concept. The states have left decisions as to the potential for planned reuse largely to the discretion of individual water users. The enlarged degree of state control which will stem from the decision in California v. United States may in some states lead to greater reuse of water. However, the fragmentation of policy which will result from state-by-state variations in the legal and institutional treatment of planned reuse may limit or slow the expansion of planned reuse in the Colorado River Basin as a whole.

The State Water Allocation Process

The second part of this article examines the actual and potential legal barriers to planned reuse which are found in the water allocation systems of each of the Colorado River Basin states. Each state depends in large part on the appropriation doctrine as the basis for water allocation determinations. A description of the major characteristics of the doctrine precedes the research findings. The discussion of legal barriers to reuse is organized according to the stage of the appropriation-and-use process at which a reuse option is contemplated and attempts to implement it are begun. Acquisition of a new appropriation right is a suitable choice of action for a prospective reuser where unappropriated, or nonutilized, water is available. In many cases, however, the stream system on which the potential reuser is or will be located is already fully appropriated and utilized, leaving little or no dependable supply for new water uses. The potential reuser must then obtain rights from existing water users through acquisition of all or part of an existing right, including the return flow. Often legal problems with the modification and exercise of the right must be solved before the water use, including reuse, may begin. Legal restraints may govern the extent of the right acquired, the degree to which the water can be put to a new use, and the overall method in which the right is exercised.

The Appropriation Doctrine. Distribution of water among competing water users in the Colorado River Basin is governed by the doctrine of prior appropriation as adopted in each of the Basin states. This method of distribution involves state administrative and judicial forums to different degrees. The right to use the water is initiated by the user appropriating the water to a beneficial use. Once obtained, this right, known as an appropriative right, is subject to limitations on its exercise through state policies or the existence of competing rights belonging to other water users.

The state-granted appropriative right in each of the Basin states is based on the assumption that water flowing in a natural watercourse is not the subject of private ownership. Private rights affecting the water are strictly usufructuary rights to take the water from the stream into physical possession for the purpose of putting it to beneficial use. Private rights of ownership do not attach to the water itself so long as it remains in the stream in its natural state. However, the right which an appropriator gains to the use of water is a private property right, subject to the same rights and responsibilities of ownership as is true for many other kinds of private property.

Although there are variations from state to state in the precise relationship between holders of senior and junior rights and between upstream and downstream diverters, the appropriation doctrine gives the following general qualities to the private water right:

- the right rests upon the application of the water to beneficial use--the time, character, and extent of the right being measured by the applications actually made in the first beneficial use to which the water is put;
- water from a common source of supply is allocated among competing users on the basis of their respective priorities, determined largely by the time of the first appropriation of water to the particular beneficial use in question;
- each right entitles its holder to allocation of all available water up to the limits of the right before the person with the next more junior right is entitled to any water at all;
- the method of exercise of the appropriative right may be freely changed to other places of diversion and use and to other purposes so long as the change does not enlarge the use in time or quantity or otherwise impair the rights vested in other appropriators;
- generally, the right to recapture and reuse water normally lost or unconsumed during the first use is included in an appropriative right, although in some states the right may become subject to the rights of the later appropriators.

These generalities concerning the appropriation doctrine govern in large part the system of water distribution in the Colorado River Basin.

Acquisition of the Right to Appropriate. There are a number of common elements required for the acquisition of an appropriative right in the CRB states. Those requirements which show a potential for affecting the viability of reuse under the state water allocation system are:

- the use to be made of the water sought must be "beneficial" under state-specific criteria;
- in all of the states except Colorado, the use proposed to be made of the water must satisfy a determination of its compatibility with the "public interest."

The concept that the public water must be used for a beneficial purpose, in accord with the public policy of the state, is fundamental in the water law philosophy of the Basin states.

All of the CRB states have some provision requiring that appropriation of water be made only for beneficial uses. In addition, some of the states have imposed further requirements on acquisition of water rights. California, for instance, in a 1928 constitutional amendment, added the requirement that the use made of public water must also be "reasonable."[9] Nevada has a specific statutory provision requiring reasonableness in the use of water.[10] Through provisions such as these, each state imposes some restraint on the efficiency of water use, at least at the time of the acquisition of the appropriative right.

The concern addressed by this investigation is whether the individual states have either constitutional, statutory, or institutional policies which would limit the potential for reuse through a determination that it was not a beneficial use of water, or that it was not a reasonable use of the water to which the right of use was granted. Legal research indicated that no constitutional or statutory determinations would limit reuse in this manner. In fact, state water policies contain some declarations to the effect that all water should be used for the maximum benefit possible. This type of declaration appears to promote reuse where otherwise possible, as transportation losses are minimized to some extent by reuse of water in one location. Interviews with officials in state water rights administration offices have shown that no institutional biases toward viewing reuse as a nonbeneficial use of public water exist. Thus, if the use would be a beneficial one where no reuse option was considered, it would also be a beneficial use if reuse supplied a portion of the water requirement.

All of the states in the Colorado River Basin except Colorado require some review of a proposed appropriation to evaluate its compatibility with what is broadly termed the "public interest." The "public interest" determination is not one regarding the protection of existing appropriators of water; such an inquiry is made elsewhere in the appropriation approval process. The determination is an analysis of all aspects of the proposed appropriation. The statutory language requiring a public interest determination in the states is largely negative, i.e., the proposed use may be approved unless there is a determination that the use would be detrimental to the public interest.[11] Given the prevalence of state policies directed toward maximization of water use, reuse is rarely seen as detrimental.

The need to maximize the use of water resources in the CRB is well recognized, and state policies encouraging that goal are widespread. Under such policies, reuse will generally be accepted where the initiative to begin reuse is undertaken by the water user. Specific reuse options may, however, be opposed on the basis that they are not as efficient as other means of water utilization. Some state officials are cautious about acceptance of reuse options as the best form of water use without analysis of all of the alternatives and the effects of the reuse, particularly with respect to water quality. While water quality aspects do not often enter into a public interest determination because water pollution control programs rely instead on regulation of discharges, there are certainly effects to be found from a reuse application. These effects may result from the lack of return flow from the reuse. Such return flow might well have historically served as dilution water for overly saline or polluted watercourse conditions.

Colorado requires no determination of the nature of or compliance with the "public interest." The Colorado system of water allocation relies, not on an administrative approval of the proposed appropriation, but on a judicial finding that the elements for a valid appropriation exist. No specific analysis of the public interest enters into the approval or denial of the appropriation.

The plans of a new appropriator who intends to reuse a portion of the water withdrawn under a water right are rarely challenged on the basis of the choice of the reuse option. In fact, state policy would, in many cases, encourage the reuse as a means of maximizing the utilization of a scarce resource. If the water user has met the requirements for the acquisition of the right, the right to reuse water is included in the appropriation right in all states except Colorado and is subject only to the same restrictions on the exercise of a right as in the case of a water use where reuse is not present.

The position of the new appropriator is, of course, one not often found in the western states in recent years. While the mechanism for the acquisition of new appropriative rights exists and is utilized in the states of the CRB, there remains little water which is not already appropriated. This is particularly true in the states where there is no requirement that the availability of unappropriated water be shown prior to the grant of an appropriative right. The result of this approach is that many streams become overappropriated, i.e., the amounts of water claimed by prospective users exceeds the available supply by a wide margin. Under the system of prior appropriation, in some years only the water users with senior priorities will have dependable supplies of water. For many of the new water users in the CRB, a dependable water supply is an economic necessity before project development can occur. The solution most often sought is the acquisition and use of an existing appropriation right with a high priority instead of, or as a supplement to, the acquisition of new water rights. The barriers to reuse in the acquisition and exercise of a senior right are the subjects of the next two sections of this paper.

Modification and Exercise of the Appropriative Right. Usufructuary rights in water are a form of property and can be freely sold or leased by the water user. The only restraints on sale or lease of water rights which exist limit the disposition of rights held by public entities such as municipal corporations and mutual districts established for irrigation and conservation purposes. Competing water users and the state have a right to see that water obtained under the right is used properly. Once ownership of a water right has been transferred from one person to another, rules may exist which limit the options of a new holder of a water right. Both statutory and case-law restrictions govern modifications of the appropriative right which involve changes in the purpose of use of water under the right, or the location at which diversion, use or return flow entry into the watercourse occurs. The extent and severity of these restrictions depends on the rights which may have been acquired by other appropriators who rely on a related source of water, and on the existence and permissible exceptions to the rule of appurtenancy. The appurtenancy rule was established early throughout most of the West in order to curb water monopoly schemes and other practices interfering with the most beneficial use of water. The rule declares that usufructuary rights in water are appurtenant to the land upon which the water is used or, in essence, that once water is appropriated and applied to beneficial use on a particular parcel of land, the right attaches to the land and cannot be exercised apart from that land or for the benefit of other land. In actuality, however, the rule was historically subject to a number of statutory exceptions. The modern form of the rule is that, whether or not the right is "appurtenant" in theory, water rights may be transferred separately from land and used on new parcels of land. Severability has been allowed by the state through (1) abolishing or ignoring the rule entirely, (2) indulging in the legal fiction that the right, when severed, immediately attaches to the new parcel of land, or (3) providing broad administratively-controlled exceptions to the rule. If compliance with the appropriate procedures is obtained, the rule entails significant restrictions on water rights transfer in only a few instances today.

State-specific limitations on the modification of water rights are designed to ensure that changes in one water right do not harm other rights. Any change sought in a water right will be approved only if no injury will result to the vested rights of other appropriators. This "no-injury" concept is common to the states of the Basin. However, state laws vary in treatment of the rights of a potential reuser of water, when the planned reuse conflicts with competing water users. The three major variations found in the CRB states where competing rights are concerned are:

(1) An appropriation right allows diversion and consumption of specific amounts of water, and the unconsumed return flow becomes a contribution to the water source. The return flow is subject to appropriation, and the resulting water right is enforceable against other appropriators generally. The continuation of the return flow, or a substitute for it, is enforceable against the original diverter.

(2) An appropriation right allows diversion of a specific amount of water, and the unconsumed return flow becomes part of the water source. The return flow is subject to appropriation, and the resulting water right is enforceable against other appropriators. The original diverter, however, has the right to recapture and consume the return flow (provided recapture is accomplished prior to loss of the diverter's control, or dominion over the water) even if such action results in loss of the source of water by subsequent appropriators of the return flow.

(3) An appropriation right allows diversion of a specific amount of water. In some instances, unconsumed return flow does not legally become a part of the water source, and is not subject to appropriation. The original diverter, therefore, may recapture and dispose of the return flow without regard to whether other water users depend on that return flow for satisfaction of their needs.

The first variation severely limits reuse potential because most of the water rights subject to it are limited in their total water consumption by the existence of dependent junior rights. The second variation is the most common in the CRB, and allows the implementation of many reuse options. The third situation described, while rare, is the most conducive to reuse.

State-by-State Summaries of Water Allocation Law

The operation of each of these variations, in conjunction with other factors of state law affecting reuse potential, are discussed below in state-by-state summaries. A brief synopsis of the major features of the variations and the law of water allocation in each of the CRB states is given in the table below.

TABLE VII

Major Legal Factors Affecting Planned Reuse

	Artificial Water (Return Flow) Appropriable	Reappropriation of Return Flow Enforceable Against Original Appropriator
Variation (1)	Yes	Yes
Variation (2)	Yes	No
Variation (3)	No	No
Arizona	No	No
California	Yes	No
Colorado	Yes	Yes
Nevada	Yes	No
New Mexico	Yes	No
Utah	Yes	No
Wyoming	Yes	No

Summaries of state water allocation law included in this report are not intended to be comprehensive. The summaries in this section will assume that a potential reuser already holds an appropriative right suitable to supply a first use, and that reuse of the return flow is contemplated. The analysis will demonstrate to what extent the recapture and reuse of return flow is possible, and the standards which must be satisfied in order to procure administrative or judicial approval of any changes involved in implementing the reuse.

Arizona. Arizona declares the water from all sources, including flood, waste, and surplus waters, to belong to the public and to be subject to appropriation.[12] Once appropriated and put to beneficial use, however, the water is no longer subject to appropriation, but is open to recovery for sale or reuse.[13] The wastewater may be captured and used by other water users, but no property right can thus be established in the use of the water.[14] The appropriator could deprive the lower water user of his employment of the wastewater, either by preventing any waste, or by recapturing the waste or surplus water from his land and applying it to some beneficial purpose.[15] Reuse or other secondary disposition of water is under the complete control of the original appropriator of the water. Irrigation water rights are not available for transfer without a permit from the state land department, which may be difficult to obtain.[16]

California. In California all water flowing in any natural channel is public water and subject to appropriation, at least to the extent that the water is not reasonably needed for a beneficial riparian use or otherwise appropriated.[17] Thus, foreign (from another stream system) and artificial (from wastewater return flow) waters are subject to appropriation once they reenter the public watercourse, i.e., have left the producer's dominion. However, the producer of artificial water owes a lower riparian user or appropriator no obligation to continue maintenance of the artificial flow, and acquires no right as against the producer to continued augmentation of the natural volume of the stream.[18] The same is true for foreign waters.[19] In those situations where recaptured water is available, changes in the method of exercising the appropriative right must not impair prior and paramount rights[20] and it must be found that the water use would best conserve the public interest.[21] If the standards set forth controlling changes in the appropriative right are satisfied, a water user is free to recapture and reuse his return flow in whatever manner he sees fit.

Colorado. All water originating or flowing into Colorado is declared to be the property of the public and subject to appropriation.[22] Colorado courts have accepted the assumption that all flowing water (including waste and seepage waters and unconsumed water from a first appropriation), finds its way to a stream, is therefore tributary to that stream, and is subject to the existing appropriations on the stream.[23] Interception of the return flow from a water use would constitute a reappropriation of the water, and cannot be accomplished to the detriment of existing rights. All intentional increases in historic consumptive use, such as through changes in irrigation technique, are treated in a similar manner. Planned reuse under an existing water right in Colorado is thus prohibited in the absence of some compensating measure, whether in terms of water supply or money damages.

Colorado has a specific statutory exception to the above rule covering foreign water imported into a stream system.[24] The developer of such water may make any disposition or succession of uses of the water that he chooses, so long as the identity of imported water can be established from waters naturally occurring in the stream.[25] Appropriators on the augmented stream can gain no right to continuance of the imported flow except by voluntary grant or contract from the developer.[26] There is therefore no limitation on changes in the use or location of use of imported water.

Nevada. The water from all sources of water supply within Nevada belongs to the public, and is subject to appropriation.[27] Wastewater may also be diverted and used but such use is subject to discontinuance of the flow by action of the original diverter.[28] Wastewater is water which has been permitted to escape after it has served the beneficial purpose of an appropriator.[29] While the use of wastewater is not a subject of statutory appropriation unless the diversion occurs after the water has returned to a public watercourse, such use constitutes a vested right and can be protected from other reappropriators.[30] Under these tenets of Nevada law, the original diverter may recapture and reuse or dispose of his wastewater, so long as the recapture takes place before the water leaves his control. The right to reuse or dispose of wastewater applies equally to a city's recapture of wastewater within a sewage system or sewage disposal plant and any other use of wastewater.[31] Modification of the appropriative right sought in order to implement a reuse is possible, but no impairment of existing rights or detriment to the public welfare may result.[32] Restricting the amount of water subject to change to the historic level of consumptive use is often the result of these requirements.

New Mexico. All natural waters flowing within streams in New Mexico are subject to appropriation.[33] There is a distinction made, however, for artificial surface waters. Artificial waters are those which appear because of escape, waste, or drainage from constructed works, and which depend for their continuance upon some actions of man.[34] This definition encompasses most municipal and industrial uses, but would exclude irrigation return flows. Artificial waters are not subject to appropriation until the end of four years after their first appearance beyond the water user's dominion. Even though, once appropriated, such waters are the basis of a vested right which cannot be infringed by other reappropriators, the appropriation of such waters cannot acquire a right against the owner or developer compelling him to continue such supply.[35] The original diverter of what became artificial waters may recapture and reuse or dispose of the waters as he sees fit without recourse for appropriators dependent on the flow of wastewater. Any recapture of the water must be accomplished before the user's loss of control, or dominion, over the water.[36]

While recapture of artificial waters is available to potential reusers, restrictions on changes in the purpose of use, or the location of diversion or use, limit in most cases the amounts of water available for reuse in new applications. Such changes in the appropriative right are allowed only if they can be accomplished without detriment to existing rights.[37] The common method of accommodating changes to this requirement is by limiting the amounts of water involved in the change to the historic consumptive use of the original diversion.

Utah. In Utah, all waters in the state are declared to be the property of the public, and therefore subject to appropriation.[38] However, waters which have been appropriated and reduced to private control cease to be public water for the period of possession and are not subject to appropriation.[39] Thus, water which is lawfully diverted from a natural source, applied to a beneficial use, and recaptured before escaping from the original appropriator's control, is available for reuse and disposition by the original appropriator.[40] Where previously appropriated wastewaters pass beyond the control of the original appropriator, they are subject to reappropriation for beneficial use, but the reappropriator of such waters, although holding a vested right as against other subsequent appropriators, cannot require that the wastewaters continue to be available for use; the original appropriator may reuse the water or sell or otherwise transfer the right to the use of such waters, provided that the reuse or other secondary disposition of the water occurs at or within a reasonable time after initiation of the present use of water under the appropriative right.[41] Limitations on reuse of wastewater include the rule that use of water must be reasonable in relation to the reasonable requirements of subsequent appropriators[42] and the statutory requirement that changes in the purpose of use and the location of diversion or use not impair any vested right to the use of water.[43] Restriction of water right modifications, often allowing a changed disposition of only the historic consumptive use, is frequently the result of these judicially and statutorily-created limitations.

Wyoming. The water of all natural streams, springs and lakes is declared to be the property of the state,[44] although state ownership is subject to a trust in the form of the right of appropriation for beneficial use.[45] Once the water is appropriated, however, the water, including seepage, waste, or unconsumed water from the original diversion, is deemed to be private water so long as it is on the lands from which it originates and thus cannot be appropriated by other prospective water users.[46] The original appropriator must lose dominion, or control, over the water by allowing its return to a natural water source before it becomes appropriable. The right which can be gained by a reappropriator is limited, as no reappropriator can compel the continuance of a waste flow on the part of a senior appropriator if the latter wishes to use his appropriated water to complete consumption.[47]

If the contemplated reuse of private waters is for a purpose or in a location different from that of the original appropriation, administrative approval must be obtained for the changes involved. The quantity of water transferred by the change may not, however, exceed the historic rate or amount of diversion or amount of consumptive use, nor decrease the historic amount of return flow or in any other manner injure the existing rights of other appropriators.[48] Unlike most other CRB states, the Wyoming restriction of transferred water to the amount of historic consumptive use is a statutory requirement rather than an administrative method for ensuring the safety of other appropriative rights.

As these brief summaries of state law show, reuse of surface water is allowable under state appropriation law in six of the seven states of the Colorado River Basin. Recapture and reuse of water following a first use may be practiced, even to the detriment of junior appropriators who depend on the return flow in order to satisfy their appropriation. However, the reuse or other secondary disposition of appropriated water must occur at or within a reasonable time after the initiation of the water use of the present water right holder. Planned reuse options attempted following the passage of a significant period of time may, in practice, be subjected to the same legal restraints as a change in the purpose or location of water use.

The right to use public waters, once obtained, is a vested property interest, subject to the same rights and responsibilities as with many forms of property. Just as with other types of property, the owner of a water right may protect his interests from infringement by other persons. Particularly with respect to water rights, though, the rights held by competing water users are not equal. Senior rights may be exercised to the limit of the right to the exclusion of junior rights, but a limitation to a reasonable method of use of the water under the senior right may be imposed for the benefit of junior appropriators. Junior appropriators initiate their rights with notice of existing rights and conditions, and in the belief that the water previously appropriated by others will continue to be used in the same manner as it is then being used. Each junior appropriator is entitled to have the source of his water remain in substantially the same condition as at the time of the appropriation. The conditions in which reliance may be placed by the junior appropriator are the regimen of the source and the quality of the water obtained.

The *regimen* of a watercourse consists of the conditions which make appropriation and use of water from the stream possible; the most important conditions are the rate and timing of flow, both from day to day and seasonally. Any alteration in these conditions which operates to the detriment of the vested rights of other appropriators may be cause for complaint to the state water rights administration agency, or it may be enjoined by the injured party in appropriate proceedings. Detrimental changes in the regimen of the water source which stem from modifications of the purpose of use or location of diversion or use under a water right are also subject to many specific statutory restrictions, as discussed earlier.

Various characteristics of water quality are important aspects of the right to use public waters. The properties of water quality which are harmful or beneficial vary from use to use and from area to area within the Basin. Each holder of a water right is entitled to enforce the continuation of water quality conditions which will substantially fulfill the purposes for which his appropriation was made. If not protected in water quality terms, the usefulness of the water right may be depreciated or even destroyed. Any use of water degrades the quality of the water to some extent, and a certain amount of degradation is included within the right granted to an appropriator. The degree of such degradation must, however, be reasonable when measured against the uses to which water is put by the injured party, not by the party who changes the quality of water. If the complaining water user can show a substantial injury resulting from changes in water quality, the user causing those changes can be enjoined from making them, or forced to pay damages for their consequences.

The ability of competing water users to enforce water quality conditions is of importance to potential reusers because of two water quality effects of reuse. The first of these effects is the addition (or concentration) of impurities in the reused water to a higher level than would result from a single use. Release of the reused water into the water source may lead to a substantial degradation in water quality. Water users injured by such a release could enjoin the release or recover damages for the injury. In addition, the release may well be regulated by water quality control regulations.

The second water quality effect of reuse is based on the consumption of waters which, when released, might serve to dilute impurities in the receiving body of water. The overall quality of the water would be diminished by the absence of the dilution effect. To the extent that such a diminution renders a substantial injury to a downstream water user, the change in return flow could be the subject of an injunction or judgment for damages. The number of instances in which this situation will arise should be limited, due to the scarcity of water users whose return flow constitutes so great a proportion of the water source that its absence would create sufficient changes in water quality to be actionable.

Groundwater. Rights to the use of groundwater in the Colorado River Basin states are controlled by a variety of doctrines, such as prior appropriation or the rule of reasonable use. Some of the states provide for appropriation of underground streams, but the rarity of such streams makes percolating groundwater, whether subject to recharge or not, the largest source of groundwater for use. Several of the states provide for designation of areas in which groundwater use has resulted in deficiencies or depletion of the source. The designation of such an area may bring increased administrative control over the use of groundwater within the area, the substitution of one form of determination of groundwater rights for another, and restrictions on groundwater use in locations apart from the point of withdrawal.

Regardless of the variations among the systems for allocation of groundwater, there are no significant differences between the treatment of a first beneficial use of groundwater and its reuse. While there are requirements that must be satisfied prior to the acquisition of the right to use groundwater, no distinction is made where planned reuse is contemplated.

Reuse of groundwater may be more feasible than reuse of surface water in the Southwest. This possibility arises because no subsequent rights to the return flow from a groundwater use can dictate the continuation or manner of disposal of the return flow. Between third parties, rights to the return flow (such as those stemming from its appropriation after entering a watercourse) are treated in the same manner as appropriation rights to surface waters.

Innovations

Legal innovations designed to implement planned reuse of municipal wastewater can be suggested on two levels. First, methods of adapting potential planned reuse options to the requirements of the various water allocation systems can be undertaken. Such methods are also employed during the initiation of non-reuse water uses. In many instances, reuse options can be readily accommodated within the existing allocation system. Second, alterations of the allocation framework intended to promote or encourage reuse might be undertaken. When suggesting such alterations, however, care must be taken to balance the benefit gained toward the most efficient water resource use by increasing the degree of planned reuse against the damage caused to the integrity of the water allocation system as a whole.

The majority of planned reuse options can be accommodated within the existing allocation systems through one or more tactical solutions to the systemic barriers designed to protect the rights of other water users. The most promising of such solutions are exchange agreements and outright purchase of existing water rights; both require permission of some state authority to make modifications of water rights.

Purchases of existing water rights can be made in most situations in order to minimize challenges to reuses by other appropriators. Proceedings will remain necessary for modification of the water right, but their complexity and consequent cost may be reduced. Buying water rights may not be economically viable because of the low yield from the rights purchased. Significant limitations may be imposed on the amount of water actually available for use in a new application, in order to protect existing holders of water rights. A purchase of a right described as 10 cubic feet per second (cfs) for a certain period of the year may, for example, yield only 6 cfs after historic consumption is determined.

The exchange of wastewater return flows for fresh water destined for non-municipal use under another water right alleviates the damage other water users might allege as a result of the reuse. However, this method of planned reuse is heavily dependent on the particular situation in which it is proposed. Both the geographical and the legal circumstances must be appropriate, and acceptance by a great number of parties may be necessary. In one example, the City of Northglenn, Colorado has reached an exchange agreement with the Farmers' Reservoir and Irrigation Co. (FRICO). The city will "borrow" water from the FRICO supply, treat it, use it, treat it again and return it to the farmers for agricultural purposes. After municipal use, a new city treatment plant will remove bacteria and heavy metals but leave phosphorus, nitrogen and potassium as agricultural nutrients. Northglenn will augment the water borrowed by 10 percent from collected storm runoff and deep wells owned by the city, as a form of interest on the borrowed water.[49]

The existing framework for water allocation in the southwestern states could undoubtedly be altered in order to encourage reuse of municipal wastewater. Statutory or regulatory preferences for reuse projects could be established or special assistance could be arranged within the water allocation system for planned reuse proposals. In practice, however, the barriers to planned reuse caused by state water law restrictions are not unreasonable ones, and it would be unwise to amend them. The damage to a long-entrenched and accepted system of water rights discipline could far outweigh any increased use of water resources to be gained by stimulating planned reuse.

Conclusions

Planned reuse of municipal wastewater shows a significant potential for increasing available water supplies in selected situations. In many instances in the arid Southwest, however, municipal reuse of wastewater would operate only to transfer the benefit of the water use from one user (the downstream appropriator who would use the municipal wastewater if no reuse were implemented) to another (the municipality or a third party to whom is transferred the right to reuse the wastewater). Planned reuse is most appropriate where its implementation will result in increased beneficial use of low quality waters or in a reduction in transmission losses.

Legal restraints on planned reuse are generally limited to providing protection for the rights of other water users. Reuse is practicable in a legal sense unless the reliance of junior appropriators on municipal return flow intervenes, after passage of a reasonable time for the implementation of a reuse. Colorado is the exception to this rule; the reuse of all water except that imported to the stream system is prohibited. Planned reuse of municipal wastewater is in most states free from allocation-oriented restraints to the extent that the wastewater is derived from groundwater sources.

Legal innovations to implement or promote reuse can be contemplated on the level of individual reuse options or of the allocation system as a whole. So long as methods are available to accommodate individual reuse projects, changes in the water allocation system as a whole in order solely to promote reuse should be carefully analyzed before implementation. There is a significant danger that the goal of increased use of water resources would be more greatly harmed by damage to the integrity of the water allocation system due to hasty change than it would be aided by the stimulation of planned reuse.

ACKNOWLEDGEMENT

This paper is derived in part from a University of Denver Research Institute study conducted in 1978. The study was funded by the Office of Water Research and Technology, U.S. Department of the Interior, under Agreement No. 14-34-0001-7816.

REFERENCES

1. Colorado River Compact, signed November 24, 1922, 45 Stat. 1057, 1064 (1928) *reprinted in* 70 Cong. Rec. 324 (1928).

2. Treaty between the United States of America and Mexico respecting utilization of waters of the Colorado and Tijuana Rivers and of the Rio Grande, Feb. 3, 1944, 59 Stat. 1219 (1945) T.I.A.S. No. 904; Protocol on the Implementation of the Treaty of Feb. 3, 1944, Nov. 14, 1944, 59 Stat. 1262 (1945).

3. Boulder Canyon Project Act (Act of December 21, 1928), Pub. L. No. 70-642, 13(a), 45 Stat. 1057 (1928).

4. Permanent and Definitive Solution to the International Problem of the Salinity of the Colorado River (Minute No. 242, International Boundary and Water Commission of the United States and Mexico), August 30, 1973, United States of America and the United Mexican States, T.I.A.S. No. 7708.

5. For this purpose, consumptive use is defined by the United States Supreme Court as diversions from the stream less such return flow as is available for consumptive use within the United States or in satisfaction of the Mexican Treaty obligation.

6. No. 77-285 (July 3, 1978), 46 U.S.L.W. 4997 (1978).

7. *Dugan v. Rank*, 372 U.S. 609 (1963).

8. TRELEASE, FRANK J. Federal-State Problems in Packaging Water Rights. Water Acquisition for Mineral Development. Rocky Mountain Mineral Law Foundation, Denver, Colorado (1978), Paper 9, p. 4.

9. Cal. Const., art. XIV, § 3.

10. Nev. Rev. Stat. § 533.060 (Supp. 1973).

11. Ariz. Rev. Stat. § 45-143 (A) (1956); Nev. Rev. Stat. § 533.370 (3) (Supp. 1973); N.M. Stat. Ann. § 75-5-6 (1968); Utah Code Ann. § 73-3-8 (1968); Wyo. Stat. § 41-203 (1957).

12. Ariz. Rev. Stat. § 45-101; 45-141 (1956).

13. *Brewster v. Salt River Valley Water Users' Association*, 229 P. 929 (1924).

14. *Wedgeworth v. Wedgeworth*, 181 P. 952 (1919).

15. *Lambeye v. Garcia*, 157 P. 977 (1916).

16. Ariz. Rev. Stat. § 45-172 (1956); While obtaining the approval of one irrigation district for the transfer of irrigation water rights may be difficult enough to obtain, one interpretation of the law would demand the approval of any irrigation district in whose watershed a transfer is contemplated before the state land department could consider an application for a transfer permit.

17. Cal. Water Code § 1201 (West 1971).

18. *Stevens v. Oakdale Irr. Dist.*, 90 P. 2d 58 (1939).

19. *Bloss v. Rahilly*, 104 P. 2d 1049 (1940).

20. *Pasadena v. Alhambra*, 207 P. 217 (1949); Cal. Water Code § 1701, 1702 (West 1971).

21. Cal. Water Code § 1255 (West 1971).

22. Colo. Rev. Stat. § 37-82-101 (1973).

23. *Peterson v. Reed*, 369 P. 2d 981 (1962).

24. Colo. Rev. Stat. § 37-82-106 (1973).

25. *City and County of Denver v. Fulton Irrigating Ditch Co.*, 506 P. 2d 144 (1972).

26. *Id.*

27. Nev. Rev. Stat. § 533.025 (Supp. 1973).

28. *Ryan v. Gallio*, 286 P. 936 (1930).

29. Nevada Attorney General's Opinions No. 98 (Jan. 22, 1944).

30. *Ryan v. Gallio*, 286 P. 936 (1930).

31. Nevada Attorney General's Opinions No. 98 (Jan. 22, 1944).

32. Nev. Rev. Stat. §§ 533.370 (1), (3) (Supp. 1973).

33. N.M. Stat. Ann. § 75-1-1 (1968).

34. N.M. Stat. Ann. § 75-5-25 (1968).

35. *Id.*

36. *Hagerman Irr. Co. v. East Grant Plains Drainage District*, 187 P. 555 (1920).

37. N.M. Stat. Ann. §§ 75-5-22, 75-5-23 (1968).

38. Utah Code Ann. § 73-1-1 (1968).

39. *Tanner v. Bacon*, 136 P. 2d 957 (1943).

40. *Smithfield West Bench Irr. Co. v. Union Central Life Ins. Co.*, 195 P. 2d 249 (1948).

41. *McNaughton v. Eaton*, 242 P. 2d 570 (1952).

42. *In re Water Rights of Escalante Valley Drainage Area*, 348 P. 2d 679 (1960).

43. Utah Code Ann. § 73-3-3 (1968).

44. Wyo. Const., art. 8, § 1.

45. *Lake DeSmet Reservoir Co. v. Kaufmann*, 292 P. 2d 482 (1955).

46. *Binning v. Miller*, 102 P. 2d 54 (1940).

47. *Bower v. Big Horn Canal Association*, 307 P. 2d 593 (1957).

48. Wyo. Stat. § 41-4.1(a) (1957).

49. DEDMON, JONATHAN. Northglenn Hopes to Borrow Water From Farmers. Rocky Mountain News [Denver] 54 (June 3, 1977).

EDUCATIONAL AND SOCIAL FACTORS AFFECTING PUBLIC ACCEPTANCE OF RECLAIMED WATER

B. H. Olson
J. A. Henning
R. A. Marshack
M. G. Rigby
Environmental Analysis
Program in Social Ecology
University of California, Irvine
Irvine, California

INTRODUCTION

Increased demand for water and the recent drought conditions in the State of California have emphasized the importance of water reclamation programs to augment diminishing water supplies. An additional benefit of renovating wastewater is the protection of the environment from pollution.

Before optimal use of this resource can be realized the public's refusal to accept reclaimed water for household purposes must be ameliorated. Previous studies have shown that the public is favorable toward low order renovation projects while opposed to higher order uses such as swimming, laundry and drinking (1, 2). The public resistance to accept reclaimed water for drinking is supported by public health concerns over the presence of chlorinated organics, pathogens, trace elements and the reliability of purification plants (3, 4).

Other factors which also have been shown to be important elements of public acceptance of reclamation projects include: Educational level (4, 5), sex (4, 5), familiarity with reclamation programs (5) and certain psychological variables (aversion to the unclean and human waste, psychological repugnance and health consciousness) (4-7).

The present investigation was undertaken to assess the importance of the nature and duration of education, psychological variables and economics and basic water supply knowledge on public attitudes toward water reuse. In order to evaluate the influence of ongoing reclamation projects two communities, one without and one with reclamation programs were selected as study sites.

METHODS AND PROCEDURES

The study populations were randomly selected from the City of Irvine, California which is a community actively involved in water reclamation projects, and the City of Anaheim which is not involved in any water reclamation projects. 500 residents of each city were selected by the random number technique from a telephone directory specifically compiled for that city. Questionnaires, and stamped return envelopes, were mailed to the study population. Reminder telephone calls were used to increase the response rate.

In an attempt to assess an individual's knowledge of, and attitude toward, reclaimed water, the questionnaire was designed using the funnel approach. The survey began with a general set of questions aimed at revealing an individual's understanding of water resources, and eventually narrowed down to specific attitudes toward reclaimed water, treatment processes involved in water reclamation and the respondent's perception of drinking water quality.

A third section of the survey ascertained the subjects' attitude toward perceived costs and benefits of using reclaimed water, assessing the impact of recent drought conditions in California on attitudes, and the perceived knowledge of supply and demand of available water resources within the state. Attitudes toward reclaimed water as they are affected by psychological variables were also examined using a seven-point Likert scale. These variables were aversion to the unclean, compulsiveness and health risk education in childhood.

Three Thurstone scales measuring participants' responses toward the use of reclaimed water for swimming, laundry and drinking were utilized (1). The Guttman scale which examined 25 uses of reclaimed water was also incorporated in the survey (1). Prior to completing the Guttman and Thurstone scales, the respondents were provided with a definition of reclaimed water to aid in their understanding of the term.

RESULTS

Of the 980 questionnaires mailed to the randomly selected population, 284 were returned by respondents and 133 were returned undelivered. Thus, the initial response was 33.5%. Forty of the returned questionnaires (4.6%) were determined unusuable due to incorrect completion, incompletion or change of residence. Therefore, 244 questionnaires (28.8%) were suitable for analysis. The data were analyzed using Pearson Moment Correlation Coefficients, One Way Analysis of Variance and Student T-Tests.

Demographic Information

Of the 121 respondents from Irvine, 47 were males, 70 females and 4 did not designate their sex. 63 males and 60 females composed the study sample for the city of Anaheim. The demographic characteristics of the respondents from both cities are shown in Table 1. The surveyed population in Irvine was younger than that in Anaheim. In addition, Irvine was the more affluent community with 42.0% of the population earning $31,000 or more per year compared to 12.2% of the Anaheim population. Over 50% of the respondents from Anaheim had lived in that city for more than 9 years, while the majority of Irvine respondents had been residents of their city less than 6 years

In Table 2 the results of one way analysis of variance indicate that there was no significant difference in acceptance of reclaimed water for swimming, drinking and laundry between Anaheim and Irvine respondents.

The results of the Guttman scales for acceptance of various uses of reclaimed water are shown in Table 3. These data indicated that there were differences between the responses of individuals surveyed in the two cities, these differences are especially noticable in the first 4 items shown in Table 3. Also Anaheim respondents viewed irrigation of crops more negatively than individuals surveyed in Irvine.

Effect of Education on Water Reclamation

Subjects ranked in order, their sources of general and water resource knowledge (Figure 1). The results indicated that though 78% of the respondents viewed formal education as the major source of knowledge, only 17% of the respondents surveyed answered that formal education contributed to their knowledge of water resources. After education, television was the major source of general information awareness, followed by newspapers (46%) and magazines (16%). The major sources for water resource information were newspapers (64%), television (43%), water district publications (36%), and magazines (31%). These data indicated that individuals often used several sources for information gathering and that special interest items were covered more frequently in specialized publications e.g., magazines.

Of the educational levels achieved (Table 4), respondents holding a doctorate or masters degree were found to be more accepting of reclaimed water for the higher uses (i.e., swimming, drinking and laundry) than the respondents who were less educated. Individuals who had attended or graduated from college were less favorable toward the use of reclaimed water for swimming and laundry than high school graduates. However, the former group was more accepting of reclaimed water for drinking than the latter. Education (Figure 2) emphasizing the sciences (biology, health/medical sciences, physical sciences and social sciences) led to a greater level of acceptance of rennovated wastewater.

Education also had a strong influence on the perceived severity of the 1975-1977 California drought (Figure 3). It should be noted that most respondents believed California had experienced a drought (Likert ±=5.5). As the level of education increased, the perceived severity of the drought also increased. Further, regardless of educational level women were less concerned by the drought conditions than men.

Effect of Respondent's Sex on Acceptance of Reclaimed Water

The data in Figure 4 indicate that women have a stronger resistance to the use of reclaimed water than men. There was a significant difference ($\alpha < 0.05$) in acceptance between men and women college graduates. In this educational category men were more accepting of all three uses of reclaimed water (swimming: d.f.=67, T=2.1; drinking: d.f.=66, T=2.1; and laundry: d.f.=67, T=2.1) than females. However, at the other educational levels no significant differences were found between men and women.

Effect of Psychological Variables on Acceptance of Reclaimed Water

Three psychological variables were examined: aversion to the unclean, warning of health risks in childhood and faith in technology (Table 5). Individuals who were repulsed by unclean conditions were also opposed to using reclaimed water. Additionally, respondents who had been warned of health risks as children were negatively disposed to the use of renovated wastewater as adults. Faith in technology by the population surveyed correlated favorably with using renovated wastewater.

Effect of Economics on Acceptance of Reclaimed Water

Respondents who believed that Southern California has a limited water supply, also felt that it was cost effective (α=0.001) to have a water reclamation plant in their city (r=0.46, N=244). Individuals who believed water reclamation was cost-effective were also in favor (α=0.001) of swimming (r=0.16, N=236), drinking (r=0.20, N=232) and laundry (r=0.21, N234) uses of reclaimed water.

DISCUSSION

The results of this study and other previous investigations (8, 9) indicate that acceptance of renovation projects within a city is not dependent upon the presence or absence of those projects (Table 2). However, as shown in Table 3, the repondents in Anaheim, a city without reclamation projects,are less favorable than those in Irvine,a city with reclamation projects,to many uses of reclaimed water. It is interesting to note though California had experienced severe drought conditions since our 1976 Irvine Study, there has been little change in Guttman responses (Table 3) toward uses of renovated wastewater in that city (3-5, 9). Comparing the three studies in communities with reclamation projects (5,9) which have been conducted using the Guttman scale developed by Bruvold (9), the greatest change over the past seven years is a slight softening in opposition toward the higher order uses of renovated wastewater. However, it appears that currently there is a higher negative response rate to uses of reclaimed water in non-project cities than was previously reported (8, 9).

The results of this investigation suggest that formal education could be of great value in promoting public awareness of water resource issues. However, findings also indicate that this educational mode is not utilized due to either lack of suitable curricula or individual course selection preferences. It appears that the most productive manner to educate the public at large concerning water management issues in through television and formal education.

As previously reported (5, 7) men are more favorable toward reclamation projects than women. However, this study indicates that the effect of gender on the acceptance of reclaimed water still requires clarification, because only among college graduates is a statistically significant difference in acceptance demonstrated between sexes.

The educational background of respondents is an important factor in acceptance of reclaimed water. Interestingly, individuals formally educated in the sciences (social or natural) were more favorable to use of reclaimed water than those educated in the humanities or business.

This investigation confirms the result of our earlier study (5) which reported that certain psychological attitudes such as "aversion to the unclean" affect an individual's acceptance of renovated wastewater for swimming, drinking and laundry uses. In addition, the present study determined that repeated childhood exposure to health risks had a negative influence on acceptance of reclaimed water. On the otherhand, faith in technology was found to favorably affect a person's attitudes. Therefore, public acceptance of reclaimed water could be achieved by reducing through health education, the negative impacts of childhood health risks as well as unclean environments and by increasing public awareness of the benefits of technology.

Perceived economic benefit also increased the respondent's acceptance levels. This information suggests that it would be worthwhile to emphasize the economic benefits of reclamation projects to the community or individuals involved.

The results of this study indicate that a wide variety of reuse practices are readily acceptable to communities and suggest that water resource managers will meet with little resistance in implementing low order use projects. Further, these findings suggest that the best introduction to reclamation projects is through their implementation, but caution should be employed in implementation of projects which envision higher order uses such as drinking or in home use.

REFERENCES

(1) BRUVOLD, WILLIAM H. Public Attitude Toward Reuse of Reclaimed Water. Water Resources Center, University of California, Los Angeles, California (1972). p. 53.

(2) OGILVIE, J. L. Wastewater Reuse as a Water Resource - The Denver Experience. Presented at A.W.W.A. 95th Annual Conference, Minneapolis, Minn. (June 8-13, 1975).

(3) BRUVOLD, WILLIAM H. Using reclaimed water: Public Attitudes and Government Policy. *Bulletin of the Institute of Government Studies*, 17:3:1 (June 1976)

(4) BRUVOLD, WILLIAM H. Environmental Attitudes. *International Encyclopedia of Psychiatry, Psychology, Psychoanalysis and Neurology*, Volume IV, Human Sciences Press (1977).

(5) OLSON, BETTY H. & PRATTE, JANICE. Public Acceptance of Expanded Uses of Renovated Wastewater. *Progress in Water Technology*, 10:1/2:319 (1978).

(6) BAUMANN, DAUNE D. & KASPERSON, ROGER E. Public Acceptance of Renovated Wastewater: Myth and Reality. *Water Resources Research*, 10:4:667 (Aug. 1974).

(7) STONE, RALPH. Water Reclamation: Technology and Public Acceptance. *Jour. Environ. Engineering Division*-ASCE (June 1976).

(8) BRUVOLD, WILLIAM H. Belief and Behavior as determinants of environmental attitudes. *Environ. and Behavior*, 5:202 (1973).

(9) BRUVOLD, WILLIAM H. & WARD, PAUL C. Public Attitudes Toward Uses of Reclaimed Wastewater. *Jour. WPCF*, 44:9:1690 (Sept. 1972).

TABLE 1

Demographic characteristics of the Study Population

Category	Percent of Study Population Irvine (N=121)	Anaheim (N=123)
Age		
18-25	3.4	7.3
26-35	31.9	19.5
36-41	18.5	17.1
42-58	33.6	38.2
>58	12.6	17.9
Family Income		
<$11,000	6.7	13.0
$11,000-20,999	12.4	18.7
$21,000-30,999	15.7	20.3
$31,000-40,999	17.3	9.0
≥$41,000	24.7	3.2
No Response	23.1	33.3
Length of Residence		
Less than a year	2.5	0.8
1 to 3 years	35.0	21.3
4 to 5 years	25.8	11.5
6 to 8 years	22.5	15.6
9 or more years	14.2	50.8

TABLE 2

Comparison of Anaheim and Irvine Residents' Acceptance of Reclaimed Water

	Swimming	Drinking	Laundry
F-ratio	2.312	0.016	0.008
Level of significance	N.S.	N.S.	N.S.
Degrees of freedom	1,232	1,230	1,234

*N.S.-Not significant at α=0.05

TABLE 3

Percentage of Individuals from Irvine and Anaheim, California Opposed to Various Water Reclamation Practices

Usage	Irvine (N=118)	Anaheim (N=122)
Drinking water	52.9	55.3
Food preparation in restaurants	52.9	61.8
Cooking in the home	47.9	54.5
Preparation of canned vegetables	47.1	56.1
Bathing in the home	38.8	35.8
Pumping down special wells	38.0	42.3
Swimming	28.9	20.3
Spreading on sandy area	24.8	30.1
Home laundry	20.7	16.3
Commercial laundry	18.2	17.1
Irrigation of dairy pasture	13.2	16.3
Electronic plant process water	12.4	10.6
Vineyard irrigation	9.1	20.3
Irrigation of vegetable crops	8.3	21.3
Commercial air conditioning	6.6	12.2
Orchard irrigation	6.6	13.8
Irrigation of freeway greenbelts	5.8	4.1
Hay or alfalfa irrigation	5.0	10.6
Home toilet flushing	5.0	8.1
Golf course hazard lakes	4.1	4.9
Pleasure boating	4.1	4.9
Residential lawn irrigation	4.1	7.3
Irrigation of recreational parks	3.3	5.7
Road construction	2.5	4.9
Golf course irrigation	1.7	3.3

TABLE 4

Educational Level of Respondents in Irvine and Anaheim California (N=229)

Educational Level	Percent of Study Population
Attended High School	3.8
High School Graduate	16.5
Attended College	28.8
College Graduate	33.9
Masters or Doctorate	19.0

TABLE 5

Pearson Correlations of Psychological Variables Which Affect Acceptance of Reclaimed Water (N=234)

Category	Swimming	Drinking	Laundry
Warning of Health Risk during Childhood	-0.16*	-0.23**	-0.25**
Aversion to the Unclean	-0.14*	-0.23**	-0.20**
Faith in Technology	0.20**	0.32**	0.23**

*$\alpha<0.05$, **$\alpha=0.001$

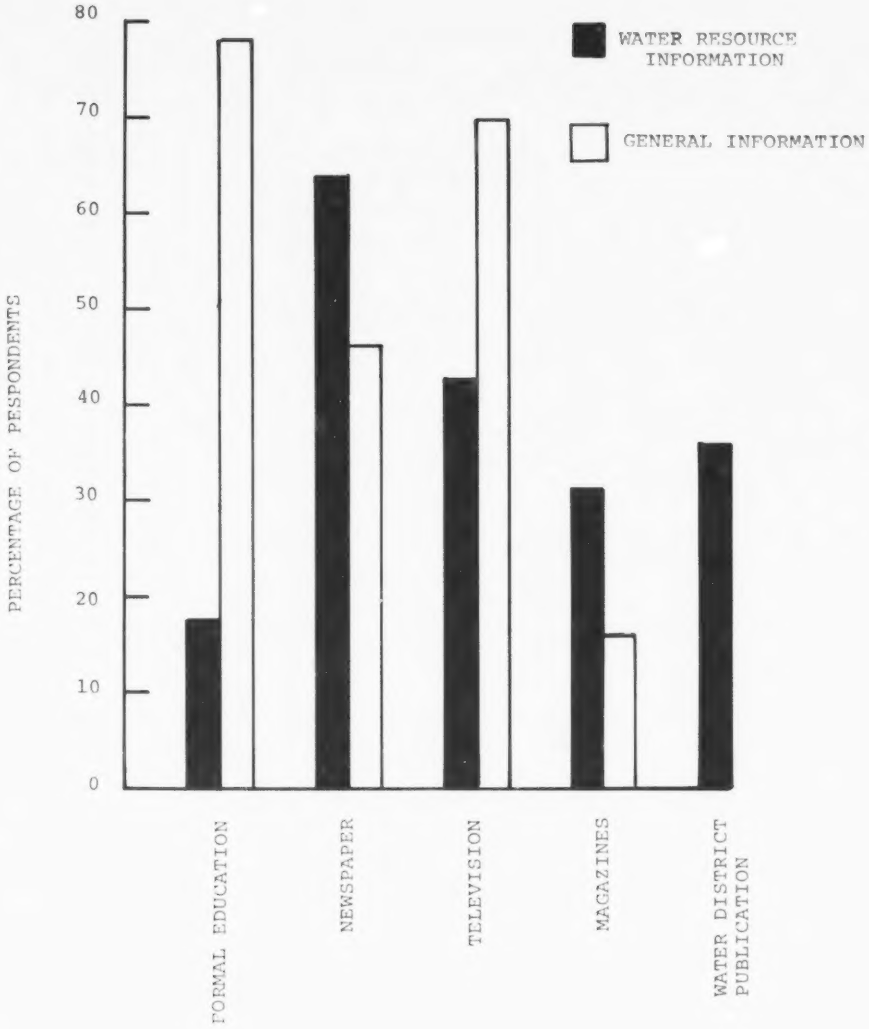


FIGURE 1

Media sources cited by respondents which contributed to general and water resource knowledge.

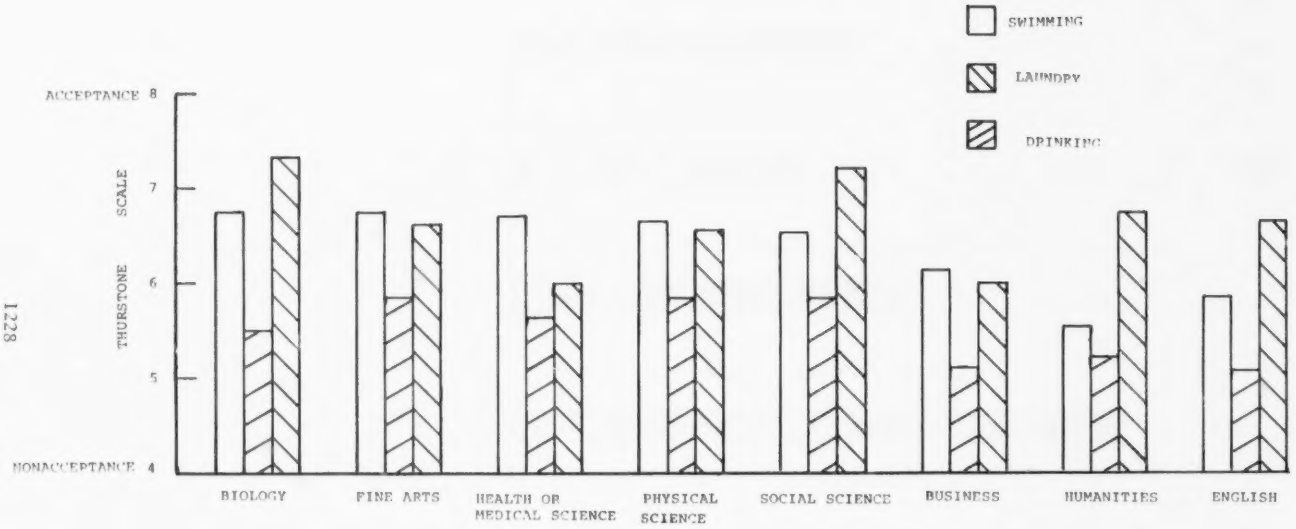


FIGURE 2

Effect of college major on respondents' acceptance of reclaimed water for drinking, swimming and laundry.

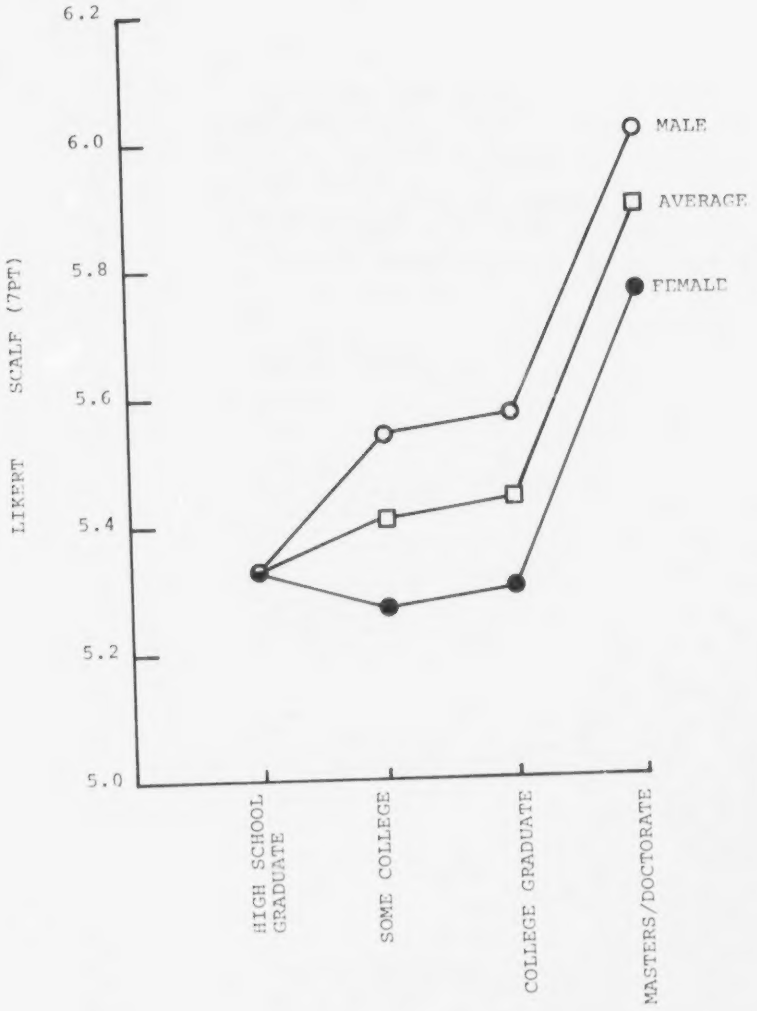


FIGURE 3

The effect of education on male and female respondents' perceived severity of the California drought (1975-1977).

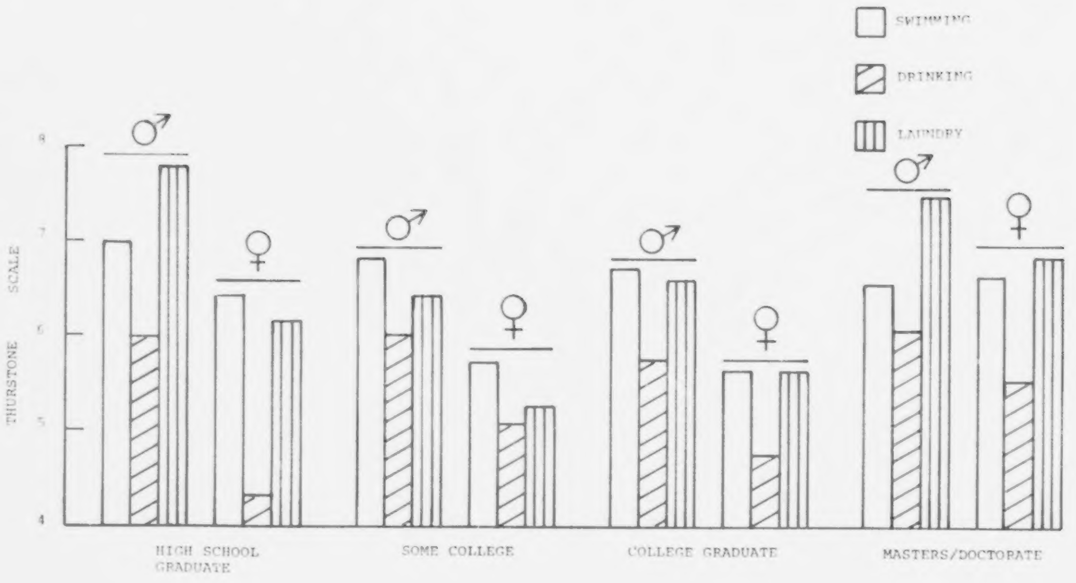
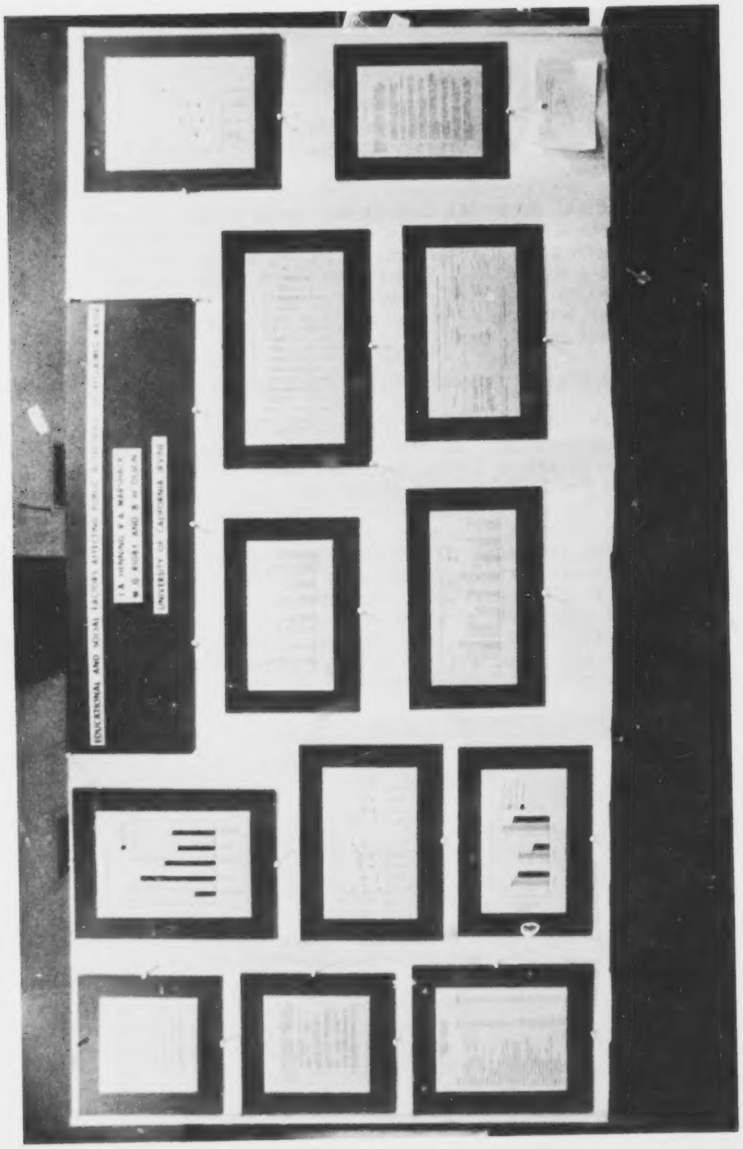


FIGURE 4

Effect of educational level and sex on the acceptance of various uses of reclaimed water. (4 = nonacceptance; 6 = neutral; 8 = acceptance).


EDUCATIONAL AND SOCIAL FACTORS AFFECTING PUBLIC
J.A. HENNING, R.A. MARSHACK
M.G. RIGBY AND B.H. DUNN
UNIVERSITY OF CALIFORNIA IRVINE

RECLAIMED WATER vs. FRESH WATER --
FROM THE USER'S VIEWPOINT

HARVEY O. BANKS
President
Water Resources Division
Camp Dresser & McKee Inc.

The four principal potential markets for large volumes of water reclaimed from municipal wastewaters are: agricultural irrigation, industrial cooling, industrial process water, and ground water recharge. In many situations, alternative fresh water sources are available. Unless very highly treated at greatly increased cost, reclaimed water would be of poorer quality than available fresh water in several areas, such as:

Salinity; Organic Content; Stable Organics; Heavy Metals, Viruses.

Users may incur substantial additional costs, per penalty costs, in using reclaimed water vs. use of fresh water because of quality differences. On a unit volume basis, reclaimed water may not be as valuable to the user as fresh water, even though both may be within the upper quality limits for the proposed user or uses. User penalty costs are site-specific. Past studies have concentrated on costs of reclaiming and distributing the water; little consideration has given to any additional internal costs to the user or user penalty costs, the prospective users might incur. Economic comparisons of reclaimed water as a source of supply vs. alternative sources must include consideration of use penalty costs. The price structure for reclaimed water should reflect user penalty costs where an alternative supply of better quality could be made available.

POTENTIAL USER PENALTY COSTS

Irrigation

1. Possible restrictions on crop types.

2. Greater leaching requirement to maintain root zone salinity within proper limit, thus a greater quantity of reclaimed water required must be applied.

3. Greater concentrations of total dissolved solids may affect crop yields. Concentrations of heavy metals and certain other constituents such as boron and cadmium may be detrimental.

4. Possible restrictions on access to irrigated lands.

5. Greater expense to control or treat and dispose of return flows.

6. Greater adverse impact on ground waters due to higher salinity.

INDUSTRIAL PROCESS WATER/COOLING WATER

1. Possible greater internal treatment costs.

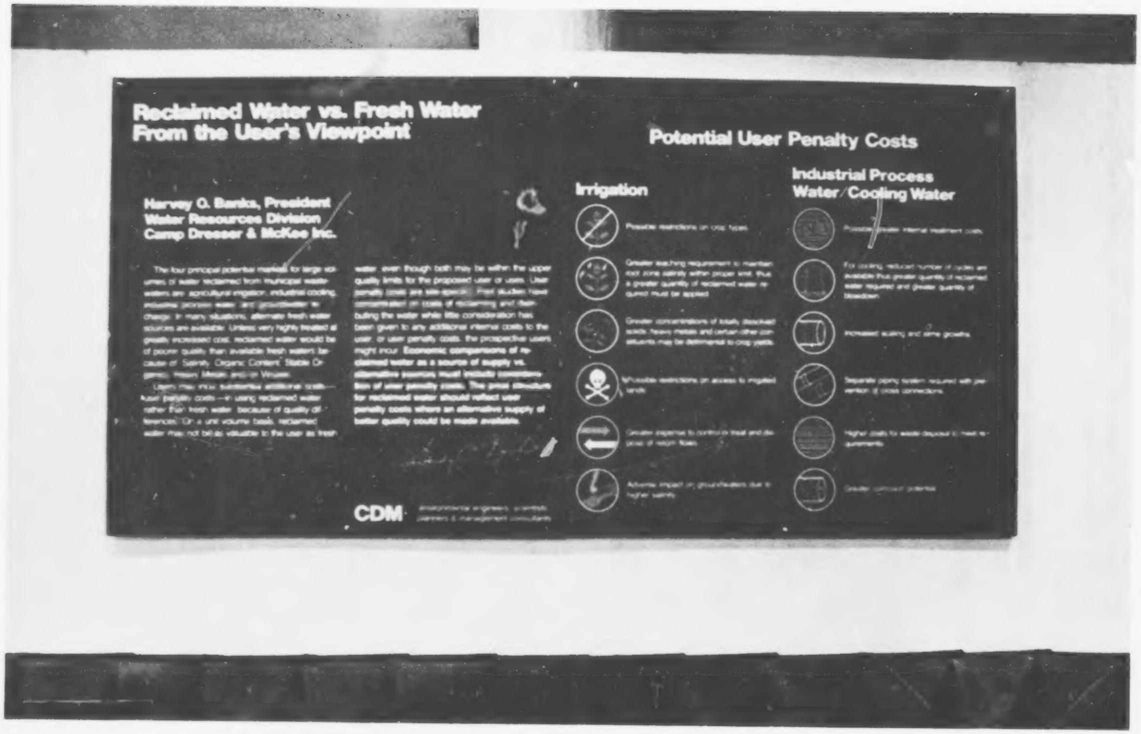
2. For cooling, reduced number of cycles are available thus greater quantity of reclaimed water required and greater quantity of blowdown.

3. Increased scaling and slime growths.

4. Separate piping system required with prevention of cross connections.

5. Higher costs for waste disposal to meet requirements.

6. Greater corrosion potential.



Reclaimed Water vs. Fresh Water From the User's Viewpoint

Harvey O. Banks, President
Water Resources Division
Camp Dresser & McKee Inc.

The four principal potential markets for large volumes of water reclaimed from municipal wastewaters are: agricultural irrigation, industrial cooling, industrial process water, and groundwater recharge. In many situations, alternate fresh water sources are available. Unless very highly treated at greatly increased cost, reclaimed water would be of poorer quality than available fresh waters because of Salinity, Organic Content, Stable Organics, Heavy Metals and/or Viruses.

Users may incur substantial additional costs—user penalty costs—in using reclaimed water rather than fresh water because of quality differences. On a unit volume basis, reclaimed water may not be as valuable to the user as fresh water, even though both may be within the upper quality limits for the proposed user or uses. User penalty costs are site-specific. Past studies have concentrated on costs of reclaiming and distributing the water while little consideration has been given to any additional internal costs to the user, or user penalty costs, the prospective users might incur. **Economic comparisons of reclaimed water as a source of supply vs. alternative sources must include consideration of user penalty costs. The price structure for reclaimed water should reflect user penalty costs where an alternative supply of better quality could be made available.**

CDM environmental engineers, scientists, planners & management consultants

Potential User Penalty Costs

Irrigation

- Possible restrictions on crop types.
- Greater leaching requirement to maintain root zone salinity within proper limit, thus a greater quantity of reclaimed water required must be applied.
- Greater concentrations of totally dissolved solids, heavy metals and certain other constituents may be detrimental to crop yields.
- Possible restrictions on access to irrigated lands.
- Greater expense to control or treat and dispose of return flows.
- Adverse impact on groundwaters due to higher salinity.

Industrial Process Water/Cooling Water

- Possible greater internal treatment costs.
- For cooling, reduced number of cycles are available thus greater quantity of reclaimed water required and greater quantity of blowdown.
- Increased scaling and slime growths.
- Separate piping system required with prevention of cross connections.
- Higher costs for waste disposal to meet requirements.
- Greater corrosion potential.

THE POTENTIAL FOR MUNICIPAL WASTEWATER REUSE BY INDUSTRY IN MASSACHUSETTS

Janice Pratte
Water Resources Research Center
University of Massachusetts

Warren Litsky
Chairman, Department of Environmental Sciences
University of Massachusetts
Amherst, Massachusetts 01003

The scarcity of suitable water sources is a problem plaguing most regions. In an effort to curb this growing problem, water conservation has become a prime issue. In June 1978, President Carter emphasized the necessity for water conservation in his water policy statement.[1] Soon afterward, Secretary of Interior Andrus named Boston as one of five areas facing drought conditions during the next ten years unless strong conservation measures are immediately taken.[2]

Even before this, Massachusetts, situated in the 'water-rich' northeastern U.S., found it necessary to prioritize water conservation efforts. The Metropolitan District Commission (MDC), a non-profit entity which provides most of the Boston metropolitan area with potable water, finds its Quabbin-Wachusett reservoir system being overtaxed.

In light of these problems, the search for alternative water sources has become intensive. The reuse of municipal wastewater effluents is a concept that holds potential for reducing strain on fresh water supplies. It does so by supplanting these sources with lower quality water where utility for this water may be indicated. The 1978 Massachusetts Water Supply Policy Statement acknowledged reuse as it highlighted conservation needs:

> "Water conservation. . .has to be enforced as a means to minimize the near-term impact of a water shortage situation. Water conservation, together with selective reuse and secondary use of non-potable supplies phased in at later dates, may also be considered as a potential supplemental water 'source' for meeting a portion of long-term water demands."[3]

In an effort to augment information particular to Massachusetts, this study examined the potential for wastewater reuse by industry as a means of water conservation. Reuse was defined as the use of treated municipal effluent in order to differentiate from recycle, which is inplant recirculation of water that has been previously used. The industrial sector was chosen as a likely candidate because water quality requirements are often less than those for potable water.

There are a number of existing features that must be examined before a wastewater reuse scheme is seriously considered. First, the planning agency must determine if reuse is essential and cost-effective. Next, the most suitable recipients of renovated effluent must be identified. Their attitudes and the attitudes of regional officials whose decisions will impact upon reuse plans must also be recognized. Variables such as these need close scrutiny long before crisis conditions cause public agencies or water customers to spend unnecessary amounts of precious capital to mend their plans. This project will help to elucidate some of these pertinent facts.

METHODOLOGY

Industrial Officials

The study was directed to the industrial plants in Massachusetts using at least 100,000 gallons per day (gpd) of municipal water. Firms with private supplies were not considered. Public service individuals whose activities impact upon water resource planning in the Commonwealth comprised the second segment of the project's target population.

The choice of industries was guided by a Massachusetts water usage study list of 'Major Water Users.'[4] The list was amended as further information was gathered, e.g., one company was contacted and subsequently included in the study upon suggestion of a participating industry official. It was arbitrarily decided that hospitals, educational facilities, and industries producing foodstuffs or sterile goods would not be considered because of their unique water quality requirements.

With the assistance of the Associated Industries of Massachusetts, the appropriate corporate officials were identified and the correct plant address obtained for each industry. A brief letter explaining the project was sent to each official on the list. A number of mailed responses were received indicating reluctance to participate, circumstances contra-indicating participation, notice that the letter was forwarded to another individual, or answers to the sample questions and agreements to participate. Approximately five days after mailing the explanatory letter these individuals were telephoned to ascertain whether participation by the particular industry was appropriate according to the criteria established at the outset of the study. This call also helped to identify the person ultimately interviewed. Numerous calls were often required before the appropriate individual, usually a plant engineer, plant manager, or manager of environmental activities was reached and an appointment time arranged.

Personal interviews guided by a questionnaire were conducted. The investigator recorded the responses. They were assured of anonymity and also that the data they might offer would be treated confidentially as it related to the specific industry. Telephone calls were made after the interviews to fill any gaps in the collected data.

A report was written after each interview. This documentation assisted in recall of the details, identified the particular setting, and highlighted aspects of the interview conversation pertinent to the research.

Responses were coded and keypunched for computer input. One-way frequency analysis of the data was accomplished using SPSS; Statistical Package for the Social Sciences.[5]

Public Officials

Twelve individuals working in public agencies concerned with water supply, water treatment, or water resource planning (Table 1) were selected to potentially participate in this study. As with the industry officials, each of these public officials was sent a letter explaining the nature of the study and requesting their participation. When telephoned, all agreed to take part and arrangements were made for a meeting at the convenience of those involved.

TABLE 1

Participating Water Resource Agencies

- Metropolitan District Commission
  - Sewerage Division
  - Water Supply Division
  - Planning
- Department of Environmental Quality Engineering
  - Water Pollution Control
  - Water Supply
- Department of Environmental Management
  - Division of Water Resources
- Water Resources Commission
- New England River Basins Commission
- Environmental Protection Agency (Region 1)
- Massachusetts Association of Conservation Commissions

Occasionally, more than one person participated in a discussion or referrals were made to individuals who were considered able to provide information. As a result, interviews were held with twenty people in twelve agencies. All participants were interviewed with supplementary information from referred individuals gathered either by telephone or in person. For the purpose of analysis, cases in which more than one individual was present during a discussion will be considered singularly as representation of one agency or division within an agency. When necessary, differences of opinion within group interviews were resolved by further discussion.

Each interview was guided by a list of questions specifically prepared for the officials in the public sector. The procedure was not formal, however, and the conversation was allowed to flow in the direction indicated by the interviewee depending upon their expertise. Nevertheless, all of the questions were addressed unless the situation indicated otherwise. In some instances questions were excluded if the individual felt uncomfortable due to a lack of technical knowledge on the subject of wastewater reuse. If they had an unfavorable attitude toward the potential of effluent reuse by industry some questions were necessarily deleted. Occasionally, aspects not covered by the prepared questions surfaced. These points were documented and will be addressed.

A report to aid in the data analysis was written for each public agency interview, consolidating the essence of the remarks and opinions expressed.

RESULTS

The following are the most significant findings pertaining to the potential of wastewater reuse by industry. A complete report and further analysis of the data will be presented at a later date.

Responses from Industry Officials

Individuals in twenty six firms who buy water to supply their needs were interviewed. There were seventeen participating industries (65.4 percent) that used more than 200,000 gpd of fresh water. Only four used the minimum 100,000 gpd as required by the study. Table 2 presents the quantities of water used by the industries represented in this study.

TABLE 2

Daily Average Amount of Water Used by Industries

Quantity (gpd)	Frequency	Percent
Approximately 100,000	4	15.4
125,000 to 160,000	5	19.2
200,000 to 650,000	10	38.5
1,000,000 to 16,000,000	7	26.9
Total	26	100.0

Industry officials were asked the quantities of water used for specific inplant purposes and the water quality requirements for each use. Twenty-one firms (80.8 percent) reported that fresh water was used for cooling purposes. Eleven of these (52.4 percent of 21) attributed more than forty-five percent of their water usage to cool machinery, condensers, or the buildings. Water quality requirements for this purpose were generally those to inhibit scale and corrosion.

The use of water for boiler feed was cited by half of the twenty-six plants involved in the study. Of these, three plants (23 percent of 13) used fifty percent or more of their incoming water for boiler makeup. Four of the firms already pretreat water for this purpose. Dissolved solids and hardness were the most frequently identified quality parameters for industries that do or do not pretreat by deionizing or reverse osmosis.

Twenty of the industries (76.9 percent) use water in a myriad of processes. In twelve instances (60 percent) more than thirty-five percent of total water intake was devoted to process use. Because the type of processes varied among the industries, the water quality requirements were just as diverse. Nine industries required pure water, nine deionized water, seven had a suspended solids limitation, seven had a pH limitation, and six were concerned with the presence of metals in the process water.

Four industries (15.4 percent) used water for the category 'other.' One of these used eighty percent of their average daily consumption for this purpose with corrosion being a property of concern. The three remaining firms used less than fifteen percent for this category.

Wherever people work, water will be used for drinking, eating, or sanitary purposes. Of the twenty-six industries participating in the study, only one (3.8 percent) consumed more than fifty percent of its total water intake for domestic purposes. Having accounted for this potable quality water, the amount used by this firm is still more than 200,000 gpd.

Degree of Acceptance

Having obtained this baseline information, industries were then asked whether they would accept the use of reclaimed water if it met their quality requirements. Eleven industries (42.3 percent) indicated they were willing to do so and six (23.1 percent) said they would accept such water under certain conditions. Nine industries (34.6 percent) indicated 'perhaps' they would agree to use reclaimed water. No industries indicated they would refuse to accept reclaimed water that met their standards.

The subject of economics was broached by asking if the company would be willing to pay the same price for reclaimed water that met drinking water standards as for the water they were presently receiving. There were nine industries (34.6 percent) that responded to this question affirmatively. Twelve industries (46.2 percent) answered negatively. Four industrial officials, representing 15.4 percent, said their companies would accept reclaimed water at the same price if certain stipulations were met.

Next, representatives were asked if their company would be willing to accept and upgrade renovated wastewater of a lesser quality than drinking water, assuming the total costs were not greater than their present costs. Of the twenty-six firms, 23.1 percent (6) claimed they were willing, and 26.9 percent (7) were unwilling. Fifty percent of the industries would agree to this proposal if certain conditions were fulfilled.

Those industries that may be paying user charges for sewerage service were asked if they would be willing to accept and upgrade reclaimed wastewater of lower than drinking water quality assuming the sum of the costs of the water, its treatment, and the sewer use charge did not exceed their present cost for water and sewerage service. There were fourteen firms that responded, half of which said they would be willing to accept renovated wastewater under such circumstances. Three of the fourteen industries (21.4 percent) were not willing to accept water as proposed, and four (28.6 percent) claimed the total costs would definitely exceed the company's present water and sewerage costs.

Figure 1 shows the number of industrial officials that indicated they would accept reclaimed water under the conditions specified in the three preceding questions. The frequency of those willing to accept reclaimed water with certain stipulations, such as a guaranteed return on investment within five years, or assurance of a quality controlled supply, changed according to the limitations attached to the hypothetical situation. More than three times as many firms attached stipulations if they accepted effluent needing further treatment than those that attached stipulations if they accepted sufficiently treated effluent.

The industrial officials were next asked what economic incentives they thought were feasible to encourage the use of reclaimed water by industry. Answers to this question included stipulations some of the respondents required in the preceding questions. More than one answer was acceptable per respondent. Table 3 groups the frequency of replies in seven categories.

TABLE 3

Economic Incentives
to Encourage Industrial Reuse of Wastewater

Incentive	Frequency
Decrease water and sewer charges	9
Favorable cost differential between reclaimed water and potable water	6
Tax exemptions or reductions	5
Profitable rate of return	4
Construction grants	3
Outside financing of dual system	2
Increase in current potable water and sewer charges	2

Industrial Attitudes Toward Water Conservation

Respondents were asked if they believed the use of reclaimed water by industry is a promising method of conserving water. A majority of industrial officials (73 percent) agreed that the use of reclaimed water could conserve some fresh water, but 88.8 percent of these (16 out of 18) attached additional statements such as: the need for economic feasibility in order for wastewater reuse to be a viable water conservation technique (4 respondents), the need for an industry to be close to the sewage treatment plant (1 respondent), the superiority of recycling as a conservation technique (11 respondents). Eight of these eleven already have water recycling systems operating within their plants and are among the 22 industries in this study (84.6 percent) that recycle water.

There were seven respondents (26.9 percent) who did not believe the use of reclaimed water by industry would sufficiently conserve water. Five of these individuals preferred other methods, such as recycling. Two other respondents did not believe there was a scarcity of water, negating the need for wastewater reuse projects.

Fourteen industries (53.8 percent) reported they have adopted a water conservation policy. Seven firms have passive policies (operation of cooling towers) and one other firm is considering a water conservation plan. Four industries were putting no effort toward reducing their water intake.

Attitudes of Public Officials

The two major concerns about wastewater reuse in Massachusetts raised during interviews with water resource officials were: (a) whether they believed the concept has potential as a means of supplying industry with some of their water needs, and (b) whether supplanting water used by industry with renovated wastewater has promise as a means of conserving fresh water supplies. The distribution of responses to these issues are identified in Table 4.

TABLE 4

Public Officials' Perspectives on Wastewater Reuse

	Yes	Undecided	No
Wastewater reuse by industry has potential in Massachusetts	6	NA	6
Wastewater reuse by industry would be an effective water conservation measure for Massachusetts	1	3	8

NA = not applicable.

Responses to the first concept were evenly divided among the twelve respondents; six disagreed there is a potential for wastewater reuse and six agreed there is a place for reuse in the future. These individuals will be considered either to be favorable or unfavorable toward the concept of reuse by industry.

Table 5 summarizes responses to this concept according to the supplemental information provided by interviewees. Those who were favorable toward reuse ranged in assuredness. Two individuals strongly believed renovated effluent could be of use in the Commonwealth. Four others expressed some faith that reuse was a viable concept needing more attention and research. One individual, very favorable toward reuse, as well as two who were slightly favorable, thought that reuse should be considered in the near future. Half of those favorable toward reuse accepted it if certain conditions were fulfilled. These requirements included proof of cost-effectiveness and safeguards such as prevention of cross-connections, adverse health effects, and other possible adverse secondary impacts. All of the individuals favorable to the concept believed that the technical capability exists to provide industries with renovated effluent that will meet their respective needs. It was noted that more intensive examination was needed to accomplish this practically and with quality assurance.

The degree to which six individuals were unfavorable toward reuse varied also. One person, slightly unfavorable, confessed to have little knowledge on the subject, but based on what was known, seriously doubted its viability. Two individuals, both engineers with many years experience in water resources, saw absolutely no potential for reuse in the Commonwealth.

Four individuals with unfavorable attitudes did concede that the situation could exist in the future where there may be a limited role to be fulfilled by renovated wastewater. Reasons ranged from imagining reuse possibly performing a very limited and restricted role to skepticism that they could assess the future with any certainty.

Only one of the six individuals unfavorable toward reuse thought that such a project could be cost-effective. A further concern expressed by three individuals in a group interview, and alluded to by three other respondents, was the inability of the MDC sewerage system to accommodate wastewater renovation without extensive engineering modifications.

TABLE 5

Characterization of Public Officials' Attitudes Toward the Potential for Wastewater Reuse by Industry

Clarifying Statement	Attitude: Favorable* Slightly	Attitude: Favorable* Very	Attitude: Unfavorable* Slightly	Attitude: Unfavorable* Very
Should be considered in the near future	2	1		
Technical capability exists	4	2		
Technical capability does not exist				1
Could occur under certain conditions	2	1	2	2
Would not be cost-effective	1		3	1
MDC unable to accommodate			2	2
Unnecessary measure			1	2
Overall attitude toward the potential for wastewater reuse	4	2	4	2

*N = 6.

One respondent noted that the Bay State is not geared to undertake such a scheme because it necessitates widespread acceptance by public officials, industrial people, and the general public. Further, the individual noted that the lack of a functioning State water resource plan would also hamper efforts to institute a reuse program.

One strongly unfavorable participant did not believe that secondary effluent, nor any effluent given reasonably priced advanced treatment, was suitable for either cooling purposes or process water for any existing Massachusetts industry. This person's unfavorable attitude toward reuse was largely based on this fact, as well as beliefs of prohibitive costs and the inconvenience necessary repiping to handle renovated wastewater entailed. This latter viewpoint was shared by three others unfavorable toward reuse as well as one respondent with a favorable attitude. Another participant cited wastewater treatment system unreliability as one reason for not accepting reuse.

The major reason for three unfavorable opinions was the belief that it was unnecessary to go to such an extreme measure to provide water. These individuals see Massachusetts having potential fresh water supplies available for at least thirty to fifty years. As the region has successfully been able to cope with increasing demands in the past, largely by constructing interbasin water transferral systems, these individuals saw no reason to doubt such success in the future. One individual estimated that undeveloped supplies are double the presently available State supplies. Further, it was once mentioned that the region has never experienced a drought severe enough to stress the water supply system beyond its capabilities according to precipitation records that date back well into the 1700's.

Associated with the unfavorable attitudes is the belief that reuse is incapable of conserving sufficient quantities of fresh water to justify its adoption. As indicated in Table 4, only one respondent in twelve agreed that reuse of effluent by industry could be an effective water conservation measure. Two of the officials interviewed did not view water conservation as an important issue.

Two respondents considered the relatively small quantities of fresh water that Massachusetts industries consume insufficient to justify reuse programs. One of these people estimated that a ten to twenty mgd demand was necessary to support reuse by industry. The other noted the typical distance of firms from sewage treatment plants as a handicap.

Others disagreed as to how conservation could best be accomplished, with eight respondents doubting that reuse by industry would effectively reduce water demand. Two individuals thought that efforts aimed toward the domestic sector were more likely to yield significant results because of the ease of implementation. Two respondents believed a recycling system utilizing cooling towers would reduce water consumption more efficiently.

One individual believed that changes in fee structures had great potential for reducing water demand and enhancing the likelihood of reuse projects. This point was also mentioned by five other participants, one neutral toward the idea of reuse as a conservation measure with the other four disagreeing that reuse would substantially conserve water.

The economic outlook of a municipal effluent reuse scheme is a major factor relating to its potential as an industrial water source. This facet was often brought up in discussions with public officials even before it was mentioned by the interviewer. It was unanimously considered that economic matters would be the most scrutinized and perplexing if and when reuse is considered in Massachusetts. Three respondents believed it unlikely that reuse by industry could ever be cost-effective.

All respondents anticipated high costs would hamper providing reclaimed water to industries. The costs to lay a dual piping system from treatment plant to an industry, with safeguards, and the costs within the plant, were mentioned as considerations of prime importance. Energy costs to pump the effluent and the costs to upgrade its treatment to the desired level were noted as pertinent. Maintenance and administrative needs were also considered costly by two respondents.

Demonstration projects were mentioned by three respondents as particularly good incentives. They noted that if a successful pilot project were to be built, raw data would be available so that officials and industries could make knowledgeable assessments.

Concern for unacceptability due to the potential adverse psychological impact reuse projects may generate was voiced by three respondents. One of these individuals suspected psychological reasons, similar to those involved with the fluoridation issue, may cause the general public to oppose the use of reclaimed water. Psychological repulsion exhibited by union workers faced with working in an environment where reuse of wastewater occurs was mentioned by another individual. The third person with a concern for psychological impacts predicted resistance from public officials working in the field of water resources.

While only one individual expressed a grave concern for the threat unknown aspects of wastewater reuse may have on the health of the general public, this same person, as well as three others, expressed the fear of cross-connection possibilities. The necessity of a dual water supply system gave these people cause to suspect that contamination of potable water, either within the industry utilizing renovated effluent or outside of the industrial site, was possible. Two of these individuals approved of the wastewater reuse concept in Massachusetts and two disapproved.

DISCUSSION AND CONCLUSION

The results of this study indicate that although there is a need for more serious consideration of wastewater reuse in Massachusetts, water resource officials are not inclined to accept it as a viable means of water conservation. Their reasons are based mostly on economics and the potential for other water supply sources. Surprisingly, the results also indicate that industries using large amounts of water are more receptive than the public sector to the prospect of a renovated wastewater use project. Industry's favorable attitude is deceiving, however, without regarding their insistance on such a project's economic viability. In other words, they do not see a need for reusing wastewater at this time, and because of this, they have attached stipulations relating to cost-effectiveness prior to their general acceptance.

It was determined that there are a sufficient number of industries that use large amounts of water in the Commonwealth to warrant closer attention to wastewater reuse in order to reduce the demand on current supplies. Unfortunately, the critical potable water shortage is unrecognized by many in the industrial sector. Although water rates are increasing, they are a small portion of an industry's capital expenditures and have not received proper attention. One plant engineer was totally convinced that the current supply was ideal: it was cheap, pure, plentiful, and would always be that way. It is apparent that comprehensive programs be instituted so that more attention is given to the recent State Water Supply Policy Statement. Further, once firms are convinced that water is a valuable and limited resource, reuse may become more acceptable.

The limited knowledge regarding wastewater reuse by some respondents in both of the groups surveyed for this study is alarming. Evidence surfaced that faith in reclamation technology is absent and the opportunity wastewater reuse offers to conserve potable water for higher uses is unrecognized.

The data imply that confusion regarding wastewater renovation masked the actual comprehension of industrial personnel. While a majority would not accept reclaimed water of the same quality as drinking water if it were to cost the same as their current supply, only one more respondent rejected rather than accepted reclaimed water of a lower grade than their current supply if the cost was the same. One would expect that more respondents would object to equal costs under the latter condition than the former. A clear understanding of the reuse concept is obviously lacking. Misconceptions must be corrected and general knowledge about wastewater reuse increased before eager acceptance of reclaimed water usage will occur. Again, an education program must be developed if reuse is to become a reality.

Where water is 'abundant' it is normal for industries to resist efforts to institute effluent reuse programs if there is a choice between renovated and potable water. Incentives must be offered. These incentives should not only be of an economic nature, such as described by the interviewees, but in the form of factual information and data from a reliable cost-benefit analysis. If this analysis is favorable, the establishment of a pilot project demonstrating reliability and cost-effectiveness is advisable. Economic and technical aspects could be presented and hopefully serve to allay the fears of the industrial sector and the general public. Proof that reuse can be practiced in the Commonwealth would enhance its image and acceptability.

It is true that New England is plentifully endowed with fresh surface water. It is also true that insufficient amounts of this water are developed to serve current and future demands, and that it takes up to two decades to activate new water resource development plans. Public officials, particularly engineers, indicated their reluctance to investigate the possibilities that wastewater reuse may have to augment current supplies and to provide a time cushion, allowing the development of new sources.

A major hope for Massachusetts' economic future is to attract industries that may have difficulties in areas with water shortages. While New England is richly endowed with running streams, planners should be careful not to rely on resources that are as yet undeveloped. Reuse can offer a reasonable and environmentally sound alternative. This study indicates that it is industry which is closer to providing the impetus for this alternative while public officials who have a duty to explore long-term solutions are still relying on traditional, multi-million dollar projects to obtain water far from the scene of its use....and waste.

A special thanks goes to the Water Resources Research Center at the University of Massachusetts for their support of this project.

Supported in part through funds provided by the U.S. Department of the Interior, Office of Water Research and Technology, as authorized under the Water Resources Act of 1964 (P.L. 88-379, as amended) through agreement No. 14-34-0001-8023.

REFERENCES

1. CARTER, JIMMY. *Presidential Documents* (June 6, 1978), p. 1044.

2. *Jour. A.W.W.A.*. 70:7 (July 1978) Update section.

3. Massachusetts Water Supply Policy Statement. Mass. Exec. Office of Environmental Affairs. Prepared by Wallace, Floyd, Ellenzweig, Moore, Inc. (May 1978), p. 178.

4. CURRAN ASSOCIATES, INC. *Water Usage Study in Communities Served by The Metropolitan District Commission.* Water Resources Research Center, University of Massachusetts, Pub. No. 51 (June 1975), p. 83.

5. NIE, N.H.,et al.. *S.P.S.S.*, McGraw Hill Book Co., New York (1975).

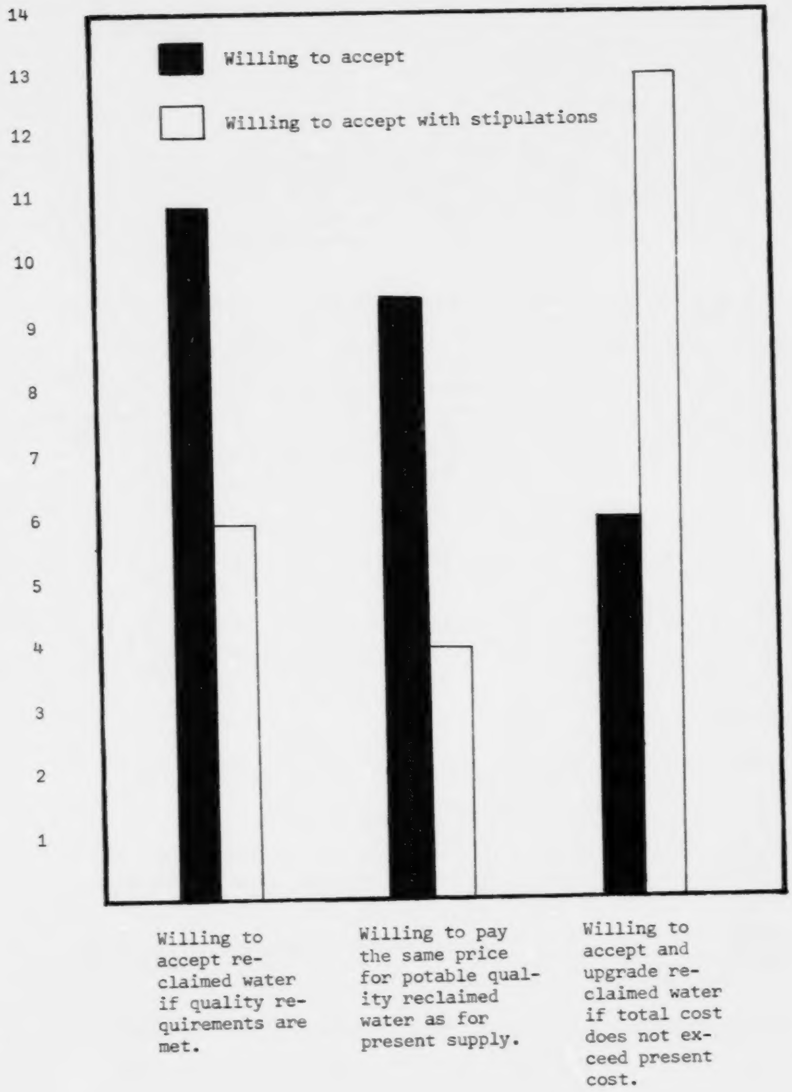


FIGURE 1

Frequency of Industries Willing to Accept Reclaimed Water

TREATMENT R & D/OPERATIONS

RELIABILITY OF ADVANCED WASTEWATER TREATMENT

Perry L. McCarty, David G. Argo, and Martin Reinhard
Professor, Stanford University; Chief Engineer,
Orange County Water District; and Research Associate,
Stanford University

Water Factory 21 is a 0.66 m^3/s (15 mgd) advanced wastewater treatment plant operated by the Orange County Water District, California to improve the quality of biologically treated municipal wastewater so that it can be used to provide the injection water for a hydraulic seawater barrier system (1). Processes included are lime treatment, air stripping, recarbonation, filtration, activated carbon adsorption, disinfection, and for one-third of the flow, reverse osmosis (Fig. 1). Since the injected water is mixed with the domestic groundwater supply, the reclamation facility must produce water that meets drinking water regulations as set forth by the California Regional Water Quality Control Board and the State Department of Health. The reliability of Water Factory 21 to meet these requirements is discussed in this paper.

Because of shortage of water in many parts of the country, there is a growing interest in reclamation of wastewater for direct potable use. Drinking water regulations have not been developed to address the special problems associated with such use. The main concern here is the relatively high concentration of organic materials and virus particles which these waters contain. The reliability to meet acceptable standards cannot be evaluated until appropriate standards are available. However, the probability of meeting given levels of specific organic materials can be determined based upon data obtained from a trace organic monitoring program conducted at Water Factory 21 over the past three years by the Stanford University Water Quality Control Research Laboratory. Such information should be useful for the eventual development of standards for direct reuse.

THE CONCEPT OF RELIABILITY

Reliability is a measure of the degree of successful performance of a facility with respect to required conditions of operation. In order to determine the reliability of a system, one must know what it is supposed to do under all relevant conditions, and what it is likely to do under all relevant circumstances. The question then is what is the probability that the achieved performance will meet the required performance?

When the required performance is specified in a list of water quality standards, then a reliable system might be defined as one which delivers water meeting these standards close to 100 percent of the time. It would not actually be necessary to always produce water meeting the standards. The quality of treated water could be continuously monitored and only that which meets the standards could be selected for delivery. Since the demand for water is generally continuous, adequate storage or another source of water must be available so that water delivery can also be reliable. The ground water basin into which Water Factory 21 effluent is injected meets this requirement.

eding page blank

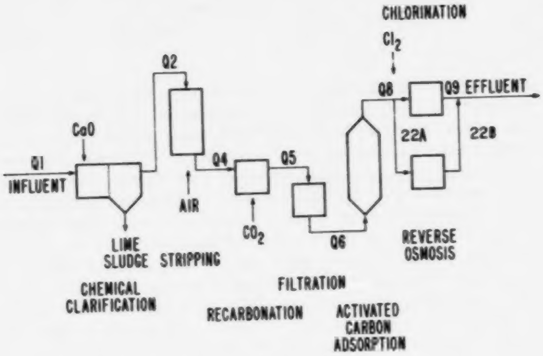


Figure 1. Flow Sheet of Treatment Processes at Water Factory 21.

Water from the treatment system need be delivered to the injection system only when it meets the required quality. This capability means that Water Factory 21 need not function in a fail-safe mode as far as equipment or process operation is concerned. If a piece of equipment fails to operate properly, then operation of the treatment system can be stopped, or the water can be wasted without injection. Also, routine maintenance can be conducted at scheduled times of the year. Use of the storage capabilities of the ground water reservoir thus gives much flexibility to the operation of Water Factory 21. It also helps reduce the cost of waste-water reclamation since no standby processes need be available.

Nevertheless it is desirable that Water Factory 21 be capable of meeting water quality requirements with high frequency in order to minimize the cost of the water delivered. The reliability of a treatment system to produce water meeting given standards can be increased in several ways as indicated in Table 1. An important aspect is the improvement in the quality of the influent water. This was achieved in March 1978 when the Orange County Sanitation District which provides the input water to Water Factory 21 changed from a relatively inefficient trickling filter system to an efficient activated sludge system for treatment of wastewaters. In addition, they removed some of the industrial wastes from the wastewaters treated for delivery to the Orange County Water District for reclamation. These changes have had a dramatic effect on the quality of the influent water and increased reliability of operation considerably. Water Factory 21 also takes advantage of the other options listed in Table 1 for increasing reliability. A reverse osmosis system was added in 1977 to treat one-third of the reclaimed water in order to meet mineral requirements for injection. Also, water from a deep aquifer can be blended with Water Factory 21 effluent to help meet mineral requirements for injection. Whenever the given water quality criteria cannot be met, the treated water is not injected, but is wasted. Also, if necessary, the efficiency of the activated carbon process can be improved by more frequent regeneration. Thus, the combination of options indicated in Table 1 are being used successfully at Water Factory 21 to improve system reliability to meet water quality requirements.

Table 1. Options for Increasing Reliability to Meet Given Water Quality Standards

1. Improve Quality of Influent Water
2. Increase Removal Efficiency of Individual Processes
3. Add Additional Processes in Series
4. Do not use treated water when standards exceeded
5. Blend effluent with other water

RELIABILITY OF OPERATION

What is the reliability with which Water Factory 21 produces water which meets water quality requirements? This was one of the major research objectives of this study. In anticipation of the time when direct potable reuse of reclaimed wastewaters becomes acceptable, it is desirable to know what is the performance of an advanced wastewater treatment plant with respect to the variety of organics of health concern which are present in municipal wastewaters. What is the effectiveness of each process in the removal of each contaminant, and how does the overall system improve this effectiveness? These questions are important in designing treatment systems for other wastewaters with other quality of influent waters and with different water quality requirements on the water to be delivered. This question required information on the frequency distribution of organics at different points within the treatment system.

The frequency distributions of over 40 inorganic and organic contaminants were evaluated at eight different sampling points at Water Factory 21 and it was found that the contaminant concentrations follow a lognormal distribution (2). An example is illustrated in Fig. 2. A lognormal distribution is characterized by the geometric mean, M, which is represented by the concentration exceeded 50 percent of the time as indicated on the graph, and the spread factor S which gives the slope of the line of best fit. The log of the spread factor equals the standard deviation of the log of the measured values. In a lognormal distribution 68 percent of the data lies between values given by MS and M/S, and 95 percent of the data lies between MS^2 and M/S^2. In Fig. 2, the K-S 10% lines represent boundaries as determined by the Kolmogorov-Smirnov test beyond which the points in the distribution should not pass if they follow the lognormal distribution (2, 3).

From a distribution such as in Fig. 2 for methylene chloride, an estimate can be made of the percentage of time that the concentration of the contaminant exceeds a given value. For example, Fig. 2 indicates that about 5 percent of the time, the influent concentration of methylene chloride exceeded 50 μg/l. Such estimates can also be made mathematically from given values for M and S. For example, it might be desired to determine the percentage of time that a given maximum contaminant level (MCL) is exceeded. First, the number of standard deviations (b) that the MCL is away from M can be determined from the following relationship:

$$\underline{b} = \frac{\log MCL - \log M}{\log S} \qquad (1)$$

A table listing the cumulative frequency function for a standardized normal distribution can then be consulted to determine the appropriate frequency value corresponding to b (3). Table 2 is a summary of a few such values of interest.

Table 2. Relationship Between Standard Deviations Above the Mean and Probability of Occurence for a Normal Distribution

Number of Standard Deviations MCL is above Geometric Mean Concentration (b)	Percentage of Time Concentration is less than MCL
0.00	50
0.84	80
1.28	90
1.64	95
2.06	98
2.33	99
3.03	99.9

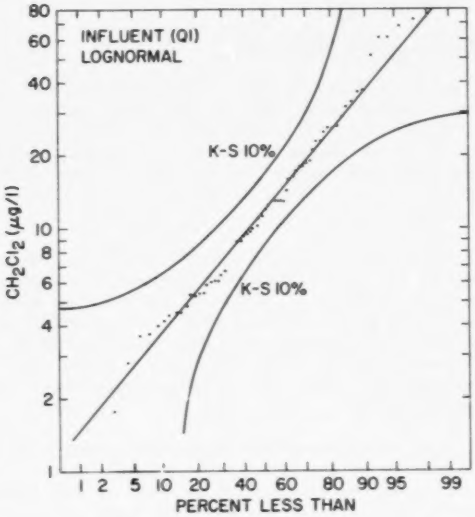


Figure 2. Example of Lognormal Distribution of Concentration for Methylene Chloride in the Influent During the Second Period.

The distributions of a variety of trace organic compounds and heavy metal concentrations at various sampling locations at Water Factory 21 were evaluated using computer graphics, and the distributions as characterized by M and S were determined as discussed previously (2). The frequency with which a given MCL or other selected value is exceeded can then readily be determined from a straight line drawn on log probability paper, by use of these two numbers, or from use of Eq. 1. The results of this analysis are summarized in the following. Because of space limitations, values for M, S, and the number of analyses upon which they are based are not presented here, but can be found elsewhere (4).

RELIABILITY IN MEETING STATE REQUIREMENTS

Water Factory 21 has been operated over three distinct periods, only two of which are considered here. During the second period from October 1976 through February 1978, the influent water was trickling filter treated municipal effluent. Since the third period beginning March 1978, the influent was changed to activated sludge effluent. Also during the third period, forced air circulation in the stripping tower has not been used.

The California Regional Water Quality Control Board and the State Department of Health have established requirements for injection water quality for Water Factory 21 (Tables 3 and 4). As additional requirements, if at any time the filter effluent turbidity exceeds 1 TU, if the activated carbon adsorption effluent exceeds 30 mg/1 COD, or if the effluent from the chlorine contact basin does not contain a chlorine residual, then the processed wastewater must be returned to the waste sewer.

Since the beginning of coastal barrier injection, there have been relatively few violations of the State regulations. Those that have occurred in the second period are listed in Table 3, and during the third period in Table 4. They are given in the last column as number of times the MCL was exceeded over the total number of samples analyzed.

In the second to the last column, the predicted percentages of time the MCLs would be exceeded as determined with Eq. 1 are given for comparison. The sampling locations are shown in Fig. 1 and are represented by the letter Q followed by a number. The predicted percentages of time the State specified MCL was exceeded during period 3 (Table 4) is over 10 percent for electrical conductivity, fluoride, and boron. In the latter two cases there is a reasonable correlation between predicted and measured values. With electrical conductivity, however, the comparison is not very good. Here the lognormal distribution was not followed well by the data, largely because electrical conductivity is continuously measured and water is not injected if the MCL is exceeded. Because of the close operational control which is possible here, the MCL need never be exceeded. This disturbs the otherwise random fluctuation in data. The predicted value is perhaps a good indicator of the frequency with which the MCL would be exceeded if such operational control were not possible.

Table 3. Comparison Between State Specified MCL for Injection Water and Actual Measured Concentrations during Period Two; October 1976 through February 1978

		Injection Water (Q10)		
Contaminant	State Specified MCL	Geometric Mean Conc.	Predicted Percent of Time MCL Exceeded*	Number of Times MCL Exceeded**
Electrical Conductivity	900 µS/cm	708 µS/cm	19	31/377
Ammonium	4 mg/l	0.6 mg/l	5	5/195
Total Nitrogen	10 mg/l	1.0 mg/l	10^{-3}	0/150
Fluoride	0.8 mg/l	0.6 mg/l	3	0/50
Boron	0.5 mg/l	0.33 mg/l	12	1/56
Chromium	0.05 mg/l	0.004 mg/l	0.6	0/55
Cadmium	0.01 mg/l	0.0004 mg/l	10^{-2}	0/55
Selenium	0.01 mg/l	<0.01 mg/l	-	0/55
Copper	1.0 mg/l	0.008 mg/l	10^{-8}	0/49
Lead	0.05 mg/l	0.001 mg/l	0.1	0/48
Mercury	0.005 mg/l	0.001 mg/l	1	1/50
Arsenic	0.05 mg/l	<0.01 mg/l	-	0/49
Iron	0.3 mg/l	0.045 mg/l	0.8	0/55
Manganese	0.05 mg/l	0.004 mg/l	10^{-3}	0/55
Barium	1.0 mg/l	0.01 mg/l	10^{-9}	0/48
Silver	0.05 mg/l	0.003 mg/l	0.1	0/47

*Based upon lognormal distribution model

**Given as m/n where m = no. of times MCL exceeded out of n samples analyzed.

RELIABILITY IN MEETING EPA PRIMARY REGULATIONS

While Water Factory 21 effluent is not used for direct potable purposes, a comparison of treatment results with the EPA National Interim Primary Drinking Water (NIPD) MCL values is desirable because of interest in advanced wastewater treatment for this purpose. In this comparison, only the influent to Water Factory 21 (Table 5) and the effluent from the basic AWT plant through activated carbon adsorption and disinfection (Table 6) are considered. Table 5 indicates that in the influent the geometric mean concentrations of only cadmium, lead, coliforms, and turbidity exceeded the NIPD MCL values. In addition, for at least 2 percent of the time, lead, mercury and fluoride exceeded the MCL values. Thus, these are the only constituents which Water Factory 21 would have to remove effectively to meet the NIPD regulations.

Table 6 indicates that the geometric mean concentration of all contaminants in the effluent from AWT treatment (Q8 or Q9) met the NIPD requirements. Also, during the second period, only chromium and

Table 4. Comparison Between State Specified MCL for Injection Water and Actual Measured Concentrations During Period Three; March 1978 through December 1978

		Injection Water (Q10)		
Contaminant	State Specified MCL	Geometric Mean Conc.	Predicted Percent of Time MCL Exceeded*	Number of Times MCL Exceeded**
Electrical Conductivity	900 μS/cm	500 μS/cm	12	0/306
Ammonium	4 mg/l	0.3 mg/l	7	0/183
Total Nitrogen	10 mg/l	0.7 mg/l	3	0/183
Fluoride	0.8 mg/l	0.57 mg/l	18	3/40
Boron	0.5 mg/l	0.38 mg/l	17	1/39
Chromium	0.05 mg/l	0.0016 mg/l	0.1	0/40
Cadmium	0.01 mg/l	0.001 mg/l	10	1/40
Copper	1.0 mg/l	0.012 mg/l	10^{-3}	0/39
Lead	0.05 mg/l	0.0016 mg/l	0.1	0/40
Iron	0.3 mg/l	0.03 mg/l	0.2	0/39
Manganese	0.05 mg/l	0.003 mg/l	0.4	1/39
Barium	1.0 mg/l	0.003 mg/l	10^{-6}	0/39
Silver	0.05 mg/l	0.001 mg/l	10^{-3}	0/40

* Based on lognormal distribution model.

** Given as m/n where m = no. of times MCL exceeded out of n samples analyzed.

mercury did not meet the requirements more than 98 percent of the time. Lognormal probability plots for cadmium and chromium during the second period are shown in Figs. 3 and 4, and are based upon the calculated values for M and S at various sampling locations. Fig. 3 indicates that lime treatment was most effective in removing cadmium, bringing the concentration well below the MCL at the Q2 sampling point. Fig. 4 also indicates that lime treatment was efficient in removing chromium, although not sufficiently to meet the MCL. Activated carbon adsorption (between points Q6 and Q8) also was beneficial, so that in the AWT effluent (Q9) the MCL was exceeded only about 10 percent of the time.

RELIABILITY FOR REMOVING ORGANIC MATERIALS

Current drinking water regulations cover few organic materials, such as the pesticides listed in Tables 5 and 6 and a current proposed MCL of 100 μg/l for trihalomethanes(5). However, there is concern over other trace organic substances which might be present in wastewaters reclaimed for potable use. Thus, there is a need for information on organic materials in reclaimed water.

Fig. 5 shows the distribution of COD at various sampling locations at Water Factory 21 during the third period of operation. It is evident that several processes are effective in removing COD including

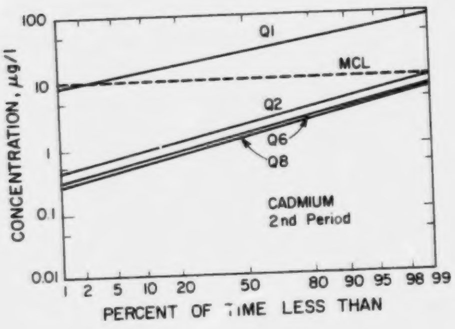


Figure 3. Frequency Distribution for Cadmium at Various Sampling Locations During the Second Period.

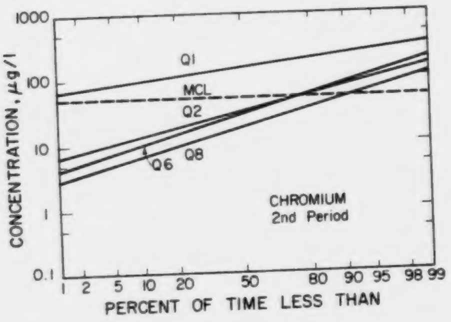


Figure 4. Frequency Distribution for Chromium at Various Sampling Locations During the Second Period.

Table 5. Comparison Between National Interim Primary Drinking Water (NIPD) Regulations and Influent Water Quality#

Contaminant	NIPD MCL	Influent Water (Q1)			
		Second Period		Third Period	
		Geometric Mean	98% of Time Less Than	Geometric Mean	98% of Time Less Than
Arsenic, mg/l	0.05	<0.005	<0.005**	<0.005	<0.005**
Barium, mg/l	1.0	0.08	0.14	0.03	0.06
Cadmium, mg/l	0.01	0.026	0.07	0.033	0.15
Chromium, mg/l	0.05	0.14	0.31	0.048	0.11
Lead, mg/l	0.05	0.02	0.051	0.007	0.017
Mercury, mg/l	0.002	0.0016	0.025	<0.001	?
Nitrate (as N), mg/l	10	<1	<1	<1	<1
Selenium, mg/l	0.01	<0.0025	<0.0025**	<0.0025	?
Silver, mg/l	0.05	0.003	0.007	0.001	0.006
Fluoride, mg/l	1.4*	1.4	2.0	1.3	1.9
Coliforms, MPN/100ml	1	89	38,000	1.6	195
Endrin, µg/l	0.2	<0.01	<0.01	<0.01	<0.01
Lindane, µg/l	4	0.2	0.9	0.14	0.22
Toxaphene, µg/l	5	<0.01	<0.01	<0.01	<0.01
2,4-D, µg/l	100	<0.01	<0.01	<0.01	<0.01
2,4,5-TP, µg/l	10	<0.01	<0.01	<0.01	<0.01
Methoxychlor, µg/l	100	<0.1	<0.1	<0.01	<0.01
Turbidity, TU	1	42	79	7	54

#Underlined values represent those exceeding MCL

* Temperature > 26.3 C°

** Based on less than 1 in 20 samples analyzed

lime treatment, activated carbon adsorption (between Q6 and Q8), and RO treatment. While the latter process is expensive, the result is water having a geometric mean COD concentration of about 2 mg/l, (equivalent to about 0.8 mg/l organic carbon) which is as low a value as found in many water supplies in the United States (7).

Table 7 is a summary of the percent probability of water meeting various COD concentrations after different stages of treatment. At Water Factory 21, the COD after activated carbon treatment (Q8) must be less than 30 mg/l, and in current operation this is reached 99.8 percent of the time. It is also met by chemical treatment 82 percent of the time and subsequent filtration increases this to 96 percent of the time. The current COD requirement can thus be met with high reliability. However, stricter COD requirement, say 10 mg/l, would necessitate more frequent regeneration of activated carbon or addition of other unit processes.

Figs. 6 through 10 show distributions for several trace organics in the influent and effluent from Water Factory 21. These compounds for the most part are contained in EPA's list of priority pollutants, and they are among the trace organics found in highest concentration in the influent waters by the current analytical methods being used. Each figure contains a group of compounds having similar structure

Table 6. Comparison Between National Interim Primary Drinking Water (NIPD) Regulations and Effluent Water Quality

		Effluent Water (Q8 or Q9) Second Period	
Contaminant	NIPD MCL	Geometric Mean	98% of Time Less Than
Arsenic, mg/l	0.05	<0.005	
Barium, mg/l	1.0	0.03	0.08
Cadmium, mg/l	0.01	0.001	0.006
Chromium, mg/l	0.05	0.02	0.09
Lead, mg/l	0.05	0.002	0.036
Mercury, mg/l	0.002	0.0017	0.012
Nitrate, mg/l	10	<10	<10
Selenium, mg/l	0.01	<0.004	?
Silver, mg/l	0.05	0.003	0.007
Fluoride, mg/l	1.4*	0.6	1.2
Coliforms, MPN/100 ml	1	0.01	14
Endrin, µg/l	0.2	<0.01	<0.01
Lindane, µg/l	4	<0.05	<0.05
Toxaphene, µg/l	5	<0.01	<0.01
2,4,5-TP, µg/l	10	<0.01	<0.01
Methoxyclor, µg/l	100	<0.1	<0.1
Turbidity, µg/l	1	0.4	1.1

#Underlined values represent those exceeding MCL

*Temperature > 26.3 °C

Table 7. Probability in Percent of Meeting Various Hypothetical COD Criteria at Different Sampling Points at Water Factory 21 During the Third Period

	Hypothetical COD Criteria				
Sampling Point	2 mg/l	5 mg/l	10 mg/l	20 mg/l	30 mg/l
Q1 - Plant Influent	0.0	0.0	0.0	0.0	1.0
Q2 - Lime Effluent	0.0	0.0	0.0	0.5	82
Q6 - Filter Effluent	0.0	0.0	0.0	9.2	96
Q8 - Act. Carb. Effluent	0.0	0.2	25	94	99.8
Q22B - RO Effluent	69	95	99.8	99.97	>99.99
Q10 - Inject. Water	1.2	35	85	99.3	99.9

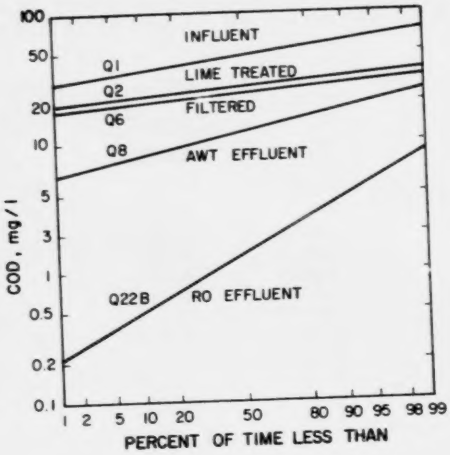


Figure 5. Frequency Distribution for COD at Various Sampling Locations During the Third Period.

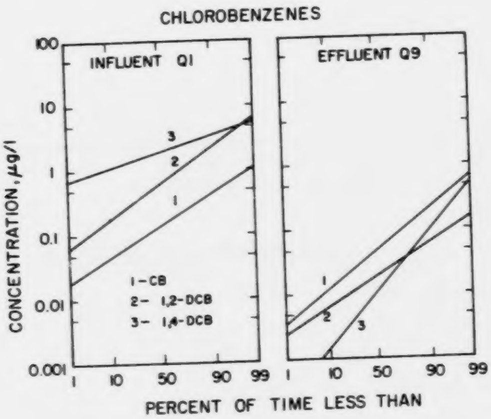


Figure 6. Frequency Distribution for Chlorobenzenes in Influent and Effluent During the Third Period. CB is Chlorobenzene; 1,2-DCB is 1,2-Dichlorobenzene, and 1,4-DCB is 1,4-Dichlorobenzene.

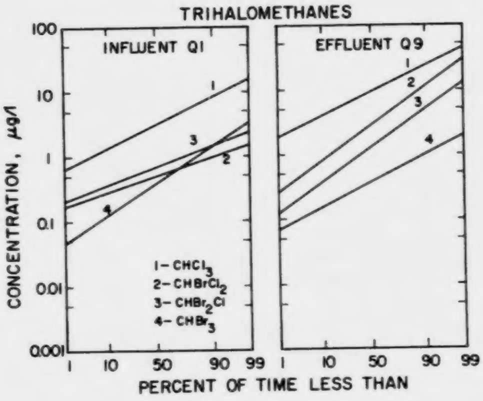


Figure 7. Frequency Distribution of Trihalomethanes in Influent and Effluent During the Third Period. Concentrations Generally Increase During Treatment Because of Formation by Chlorination.

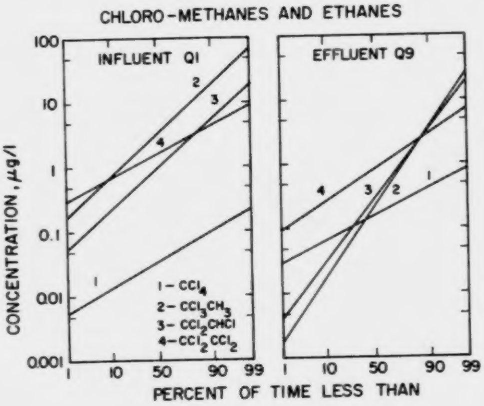


Figure 8. Frequency Distribution of Chlorinated Methanes and Ethanes Other than Trihalomethanes in the Influent and Effluent During the Third Period.

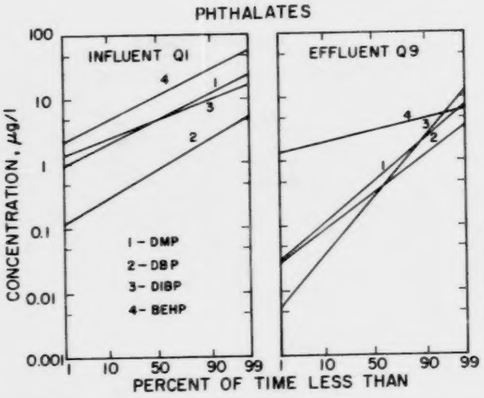


Figure 9. Frequency Distribution of Phthalates in the Influent and Effluent During the Third Period. DMP is Dimethylphthalate, DBP is Dibutylphthalate, DIBP is Diisobutylphthalate, and BEHP is Bis-(2-ethylhexyl)phthalate.

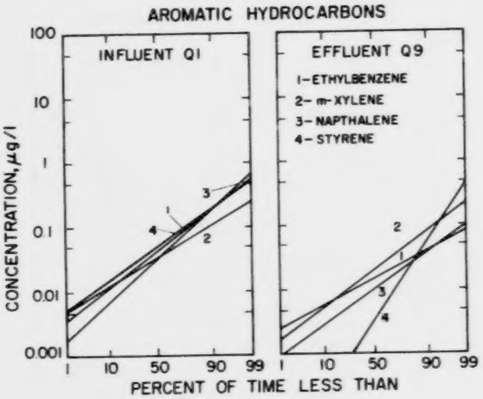


Figure 10. Frequency Distribution of Aromatic Hydrocarbons in the Influent and Effluent During the Third Period.

and similar removal properties by advanced waste treatment. The trihalomethanes (Fig. 7) were the only group with a net increase in concentration during treatment. They are formed during chlorination.

Removal of representative compounds from each of the above groups are indicated by the distributions at various sampling locations shown in Figs. 11 through 14. Chlorobenzenes (Fig. 11) are removed most efficiently by air stripping (Q2 to Q4) and activated carbon adsorption (Q6 to Q9). Air stripping is the most effective process for tetrachloroethylene (Fig. 12) and the other chlorinated methanes and ethanes, including trihalomethanes. These compounds are removed poorly by activated carbon adsorption. Phthalates, on the other hand (Fig. 13) are most efficiently removed by activated carbon. Ethylbenzene (Fig. 14) is removed partially by several processes. The influent concentration of this compound and other aromatic hydrocarbons was extremely low so that here good comparisons between processes is difficult.

While MCLs for these trace organics have not been established, it is useful to pose hypothetical values in order to determine which compounds or classes of compounds may present the most difficulties in meeting potable reuse regulations should they eventually be developed. In order to do this, hypothetical MCL values which are probably below concentrations which would cause a health effect were chosen. The hypothetical MCLs are 1 μg/l for the non-chlorinated trace organics and 0.5 μg/l for the chlorinated trace organics. These values are significantly below all NIPD MCL values for pesticides except endrin.

Tables 8 and 9 indicate the percentage of time the influent and effluent waters from Water Factory 21 exceeded these hypothetical MCL values during the second and third periods at Water Factory 21. During the second period, all of the organics exceeded the hypothetical MCL in the influent more than 10 percent of the time, while during the third period only about one-half did. This illustrates an advantage of efficient biological treatment and segregation of industrial wastes prior to advanced treatment. The data in Table 9 indicate that after treatment through activated carbon adsorption and disinfection (Q9) during the third period, only some of the chlorinated methanes and ethanes, and some of the phthalates exceeded the hypothetical MCL values more than 10 percent of the time. It is interesting to note that reverse osmosis was of little help in removing these compounds. The chlorobenzenes, aromatic hydrocarbons, and pesticides were all efficiently and reliably removed by treatment.

The chlorinated methanes and ethanes are common solvents and appear to present a special problem. They were removed most effectively by air stripping, even during the third period when forced air circulation was not used in the stripping towers. However, during subsequent treatment, contamination frequently resulted. These materials are generally used in solvents, paints, and coatings. Occasional increased concentrations at different points in the treatment plant may have resulted from use of materials containing organic solvents during normal plant maintenance. This needs further investigation. The phthalates are commonly used as plasticizers and can be leached from plastic pipe. It is also difficult to keep samples from becoming contaminated with phthalates after collection so that reliable analyses is difficult. Thus, if actual standards near the hypothetical MCL values for these materials are proposed, then greater

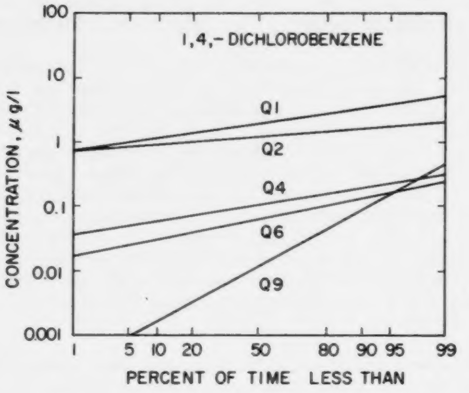


Figure 11. Frequency Distribution of 1,4-Dichlorobenzene at Various Sampling Locations During the Third Period.

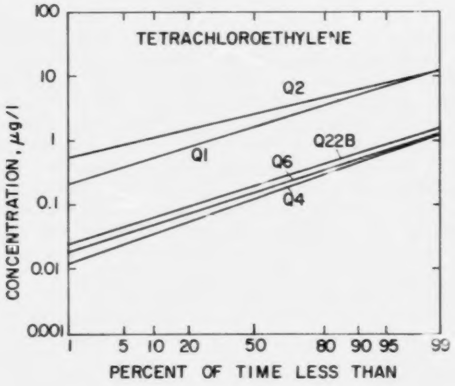


Figure 12. Frequency Distribution of tetrachloroethylene at Various Sampling Locations During the Third Period.

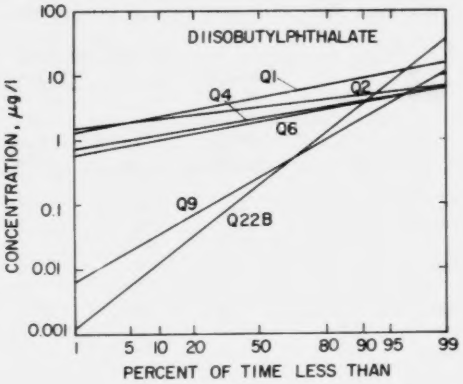


Figure 13. Frequency Distribution of Diisobutylphthalate at Various Sampling Locations During the Third Period.

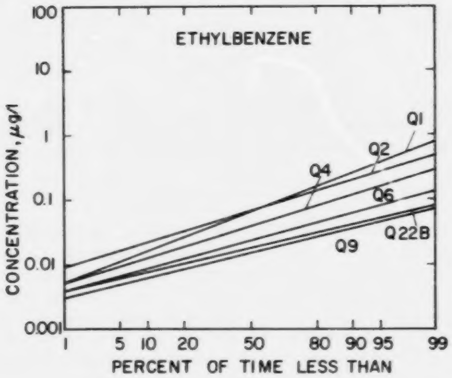


Figure 14. Frequency Distribution of Ethylbenzene at Various Sampling Locations During the Third Period.

Table 8. Percentages of Time Hypothetical MCLs for Various Trace Organics Were Exceeded During Second Period

Contaminant	Hypothetical MCL µg/l	Geometric Mean Influent Conc. µg/l	Percent of Time Hypothetical MCL Exceeded: Influent Q1	AWT Effluent Q9	RO Effluent Q22B
Methylene Chloride	0.5	17	99.99	82	-
1,1,1-Trichloroethane	0.5	4.7	98	6	-
Trichloroethylene	0.5	0.9	67	5	-
Tetrachloroethylene	0.5	0.6	54	21	-
Chlorobenzene	0.5	2.5	95	1	0.4
1,2-Dichlorobenzene	0.5	2.4	96	5	10^{-7}
1,3-Dichlorobenzene	0.5	0.7	61	6	10^{-4}
1,4-Dichlorobenzene	0.5	2.1	95	15	0.2
1,2,4-Dichlorobenzene	0.5	0.5	48	6	-
Ethylbenzene	1.0	1.4	63	0.2	10^{-4}
Napthalene	1.0	0.6	28	10^{-3}	0.2
1-Methylnapthalene	1.0	0.9	42	10^{-6}	0.7
2-Methylnapthalene	1.0	1.0	50	0.5	2.5
Dimethylphthalate	1.0	16	100	79	-
Di-n-butylphthalate	1.0	<0.5	-	20	87
Diisobutylphthalate	1.0	2.9	86	42	100
Bis-[2-ethylhexyl]-phthalate	1.0	28	100	85	86
PCB as Aroclor 1242	0.5	3.3	99.9	<12	<20
Lindane	0.5	0.2	10	<12	<20

care would be needed in the selection of materials for treatment plant process coatings and pipe lines, of materials used in normal plant maintenance, and of sampling and analytical procedures.

SUMMARY AND DISCUSSION

This study has shown that a full scale advanced wastewater treatment system was capable of good reliability in the removal of many contaminants from biologically treated municipal wastewater. Reliability at Water Factory 21 is increased through the operating philosophy which has been adapted. This plant is operated under a constant flow condition so that hydraulic fluctuations are eliminated. Significant removal of industrial wastes from municipal sewage and efficient biological treatment of wastewaters prior to advanced wastewater treatment has had major effects in reducing the concentration of some of the contaminants of concern, thus enhancing the reliability for meeting the treated water requirements. Water Factory 21 can be shut down when poor quality water is received or when desirable for routine maintenance because water need not be injected continuously for maintenance of the seawater barrier system. This increases the flexibility of operation considerably and enhances reliability in the delivery of water meeting given water quality requirements.

Another factor affecting reliability is the physical chemical system. Little time is required to adjust plant operating conditions properly after starting or stopping, or whenever desirable. In addition, the

Table 9. Percentage of Time Hypothetical MCL Values for Various Trace Organics Were Exceeded During the Third Period

Contaminant	Hypothetical MCL µg/l	Geometric Mean Influent Conc. µg/l	Percent of Time Hypothetical MCL Exceeded		
			Influent Q1	AWT Effluent Q9	RO Effluent Q22B
Carbon Tetrachloride	0.5	0.03	0.1	6	5
1,1,1-Trichloroethan	0.5	3.2	92	33	28
Trichloroethylene	0.5	1.0	71	38	40
Tetrachloroethylene	0.5	1.7	92	27*	17
Chlorobenzene	0.5	0.14	8	2	0.8
1,2-Dichlorobenzene	0.5	0.64	60	0.03	0.8
1,3-Dichlorobenzene	0.5	0.16	8	<3	0.1
1,4-Dichlorobenzene	0.5	1.8	99.5	1	0.03
1,2,4-Trichlorobenzee	0.5	0.11	12	<3	<6
Heptaldehyde	1.0	0.1	1	2	-
Heptylcyanide	1.0	0.002	0.2	0.1	-
Ethylbenzene	1.0	0.04	0.4	10^{-5}	10^{-5}
m-Xylene	1.0	0.04	0.003	0.003	0.002
p-Xylene	1.0	0.02	10^{-8}	10^{-10}	10^{-9}
Napthalene	1.0	0.03	0.2	0.001	0.001
1-Methylnapthalene	1.0	0.01	10^{-5}	<3	0.8
2-Methylnapthalene	1.0	0.01	0.1	<3	10^{-5}
Styrene	1.0	0.05	0.1	0.5	-
Dimethylphthalate	1.0	4.8	99	26	50
Diethylphthalate	1.0	0.10	15	<4	<8
Di-n-butylphthalate	1.0	0.79	39	14	56
Diisobutylphthalate	1.0	4.7	99.7	21	25
Bis-[2-ethylhexyl] phthalate	1.0	11	99.9	>99.9	85
PCB as Aroclor 1242	0.5	0.47	45	<5	<7
Lindane	0.5	0.14	10^{-7}	<5	<8

*Based upon concentration at Q8, valves at Q9 were higher due to leaching from coating on chlorine contact basin.

groundwater basin itself enhances the reliability of this type of system. The aquifer represents a very large underground storage reservoir. As the reclaimed water travels from the injection point through layers of clay and sand, some constituents may be removed from the water, which is ultimately withdrawn at points several hundreds of meters from the injection point. Other constituents will alternately adsorb and desorb, and by this process the effects of any peak concentrations will be evened out as the water moves through the groundwater system (6). For this reason, occasional maximum values which exceed requirements may not be as important as average values. Thus, the mean values could possibly be more meaningful than extremes in reclaimed effluents used for groundwater injection. These considerations should be carefully evaluated before facilities for potable reuse of municipal wastewaters are designed.

Finally, a note of caution must be expressed when considering the limited analysis presented here in relation to direct reuse for potable purposes. While the distribution of concentrations for many trace materials have been quantified in this study, a large fraction of the organic carbon remains yet uncharacterized. The possible health significance of these materials still needs to be evaluated before direct reuse of municipal wastes for potable purposes can be implemented.

ACKNOWLEDGEMENTS

This study was supported by the Wastewater Research Division, U. S. Environmental Protection Agency through Grant EPA-S-803873; the Office of Water Research and Technology, U.S. Department of the Interior through Grant 14-34-0001-7303; and the California Department of Water Resources through Grant B52353. Appreciation is extended to James Graydon, Joan Schreiner, Kenneth Sutherland, Lewis Yee, and Huong Nguyen for their assistance in sample analyses.

REFERENCES

1. McCarty, P.L., M. Reinhard, and D.G. Argo. Organics Removal by Advanced Wastewater Treatment. Proc. AWWA, Part 1, 5-3, 1-26 (May 1977).

2. McCarty, P.L. and M. Reinhard. Statistical Evaluation of Trace Organics Removal by Advanced Wastewater Treatment. WPCF Annual Conf. (Oct. 1978).

3. Benjamin, J.R. and C.A. Cornell. Probability, Statistics and Decision for Civil Engineers. McGraw-Hill Book Co., New York (1970).

4. McCarty, P.L., M. Reinhard, J. Graydon, J. Schreiner, K. Sutherland, T. Everhart, D.G. Argo. The Performance and Reliability of Advanced Treatment for Wastewater Reclamation at Water Factory 21. C.E. Tech Report No. 236, Stanford University (May 1978).

5. U.S. Enviornmental Protection Agency. Interim Primary Drinking Water Regulations - Control of Organic Contaminants in Drinking Water. Federal Register, 575b (Feb. 9, 1978).

6. Roberts, P.V., P. L. McCarty, M. Reinhard and J. Schreiner. Organic Contaminant Behavior During Groundwater Recharge. WPCF Annual Conf. (Oct. 1978).

7. Symons, J. M. et.al. National Organics Reconnaissance Survey for Halogenated Organics. Jour. AWWA, 67, 634 (1975).

HUMAN FACTORS IN DESIGN AND OPERATION OF WATER REUSE FACILITIES

ROY D. MILLER, Ph.D., P.E.
Captain, Medical Service Corps and Chief
Environmental Health Engineering Branch
US Army Environmental Hygiene Agency
Regional Division - North
Fort George G. Meade, Maryland

WALTER P. LAMBERT, Ph.D.
Major, Medical Service Corps and
Staff Sanitary Engineer
Headquarters, US Army Medical Research
and Development Command
Fort Detrick, Frederick, Maryland

INTRODUCTION

The success of wastewater reuse as a national policy will depend upon exemplary performance by reuse facilities. Performance will depend upon operability of the plant with the workforce on hand. Operability will be determined by explicit design of the interfaces between the plant operator, his machines and his management organization. Human Factors Engineering concepts are recommended for design of both interfaces. Management organization design is recommended as an explicit function executed simultaneously with facility system design. A management staff position is recommended for a sanitary engineer for the purpose of providing technical assistance to all levels of management including the plant operator.

MAN/MACHINE INTERFACE

A. Background.

Human Factors Engineering (HFE) is defined by one source as, ". . . a comprehensive technical effort to integrate all personnel characteristics (skills, training implications, behavioral reactions, human performance, anthropometric data and biomedical factors) into . . . systems to assure operational effectiveness . . .".[1] The consumer industry has applied HFE to design of recreational equipment and automobile interiors. Industrial process control panels are commonly designed through HFE. To date, wastewater treatment literature has not reflected a significant concern for the role HFE might play in plant design.

The potential importance of HFE to reuse has been demonstrated on an Army reuse development project.[2,3,4] Success of that system depended upon a long list of fatality factors. Operator skill level headed the list. A successful system had to be operable by a minimally trained draftee who would rather be doing something else. This perception led to a significant effort in control system design detailed

---

"The opinions or assertions contained herein are the private views of the authors and are not to be construed as official or as reflecting the views of the Department of the Army or the Department of Defense."

by Dr. Yang.[5] The important lesson was that satisfying engineering performance criteria was necessary, but not sufficient, for system success.

That lesson is applicable to reuse in a broader context. There is an obvious trend toward greater system complexity and sophistication. Reuse demands more and different unit processes than does conventional treatment. Tighter and more sophisticated controls are needed for reliable operation. Implications of higher operator skill levels are difficult to avoid. Demands on product quality are increasing. Reuse in particular necessitates increased demands on plant reliability and product quality assurance.[6] Disparities between engineering design capability and actual plant performance are becoming increasingly unacceptable. Demands on the plant operator can only increase. Proper design of the man/machine interface will be critical to attainment of national goals for wastewater reuse.

B. Concepts.

Three man/machine concepts are important: (1) fitting the man to the machine, (2) fitting the machine to the man, and (3) a combination of the two. Fitting the man to the machine is commonly equated with personnel training. It is realized that the plant operator is critical to producing an acceptable product. The US Environmental Protection Agency (EPA) found that the highest ranking factor contributing to poor municipal treatment plant performance was ". . . operator application of concepts and testing to process control. . .".[7] The General Accounting Office has recommended that EPA require states to conduct regularly scheduled operation and maintenance inspections at municipal waste treatment plants as a result of past poor plant performance.[8] Investment in real property for water and wastewater systems and the annual expenditure for operation and maintenance should dictate the highest level of care possible for water and wastewater systems. On the other hand, it appears that operator training and certification programs alone will not insure proper plant operation and subsequent attainment of environmental quality goals. The EPA found that even trained operators usually were not able to apply basic wastewater treatment concepts to their individual situations.[9] New approaches to operator training were recommended. An additional factor is the introduction of new technologies at an increasingly rapid pace. Robens of Woldingham remarked that, ". . . change will continue as the technology changes, and this means a continuous training process; certainly not a single training for the rest of an operator's life, but training which is continuous in order to keep pace with the new technology as it develops".[10] Unquestioned emphasis on operator training, however, presupposes (a) the existence of a manpower pool of appropriate aptitude and size and (b) essentially no bounds on the adaptability of the on-board workforce. The possibility that limits may exist to the size and quality of the manpower pool appears to worry treatment plant managers but not necessarily plant designers. The possibility that limits may exist to either the rate or extent of fitting the man to the machine concerns social theorists and psychologists but not necessarily sanitary engineers. There is at least the conceptual possibility that overemphasis on fitting the man to the machine may be a serious suboptimization within a total national pollution abatement system.

The converse of fitting men to machines is fitting machines to men, the underlying conceptual target of HFE. Notable successes have been achieved using such concepts. The telephone system is one: a system designed by geniuses for use by idiots. There are recent indications that HFE concepts may enter the sanitary engineering field in company with computers.[11,12] If HFE were to become a standard component of reuse activites, interesting conceptual implications result. HFE concepts could be applied to design of the systems with which the operator works and to the design of the operator's job. More interestingly, the introduction of HFE in company with computers and automation may reveal that the operator has been a poorly defined entity with a job description but without a job design. This is important because ideal system design starts with a clear definition of the operator and moves backward through design of the automatic control system into the mechanical process system.

Two questions remain: (1) when in the development of a system should HFE be applied, and (2) what kinds of decisions can HFE influence. There is a logic which suggests that HFE should enter during research and development of unit processes and continue through at least total system design.[13] Early on participation by human factors engineers influences important design decisions in the most cost effective manner and reduces certain cosmetic and rescue applications of HFE. HFE applications, during unit process development, implies some level of HFE involvement by the engineering research and development community. Application of HFE concepts is best made through collaboration between design engineers and psychologists. That a psychologist might influence unit process selection, equipment placement, control panel design and a myriad of other decisions may be only part of the cultural shock which the field of sanitary engineering is experiencing.

If dangers exist from overemphasis on fitting men to machines, there are conceptual dangers involved in overemphasis in fitting machines to men. A cost and performance effective compromise probably exists between the two. One implication is some degree of parity between system mechanical performance and system human performance. Both are critical to system operability, and they lie on either side of the man/machine interface.

MAN/ORGANIZATION INTERFACE

A. Background.

A Water Pollution Control Federation (WPCF) committee has recently reported seven problem areas in poor plant performance: regulations, regulators, owners, design and construction agencies, equipment suppliers, facility operators and facility management.[14] Everyone, including the taxpayer as ultimate owner of public facilities, is regarded as a contributor to poor plant performance. With everyone at fault, who can be blamed? The weakest voice in the crowd, the plant operator. Why? Because the list of problem areas appears to be a symptomatology, not an etiology. Attacks are, therefore, mounted against the highest profile symptoms, not against the underlying causes. There is every reason to expect that the list will increase unless the underlying causes of the problems are identified and dealt with.

B. Concept.

Just as the man/machine interface must be explicitly designed as the linkage between human and system performance, so must the man/organization interface be explicitly designed. The reason being that the man/organization interface is the linkage between management and operations; the WPCF symptomatology strongly implied absence or continual disruption of that linkage. Miller and Rice contended that two functions are important to the establishment of new works: (1) plant design and construction, and (2) organization design and construction.[15] Both functions must be executed simultaneously. Management and operations personnel should be involved in facility design in an effort to obviate problems and simplify operations and maintenance of the facility.[15] Similarly, design engineers should be involved in organization design and construction. One example of the failure of proper establishment of works is the apparent absence of job design as part of organization design for a treatment management organization. The result has been poorly designed jobs poorly performed at all levels of management.

A common behavior by management in this situation is the divestiture of operational decisions from levels of management above the plant operator. This makes the operator accountable for all the plant does or fails to do without benefit of organizational assistance. If jobs were designed as part of the facility acquisition process, explicit bounds on operational decisions could be expected. The plant operator would not be assumed to make all decisions. Instead, he would make all operational decisions below a certain technical threshold, and would call upon technical staff assistance within the management organization beyond that threshold. The plant operator currently does not receive technical staff assistance because organization design does not include appropriate technical staff. The appropriate role of technical staff is to provide professional assistance to all levels of management.[16] The plant operator is one level.

Organization design for technical staff appears to be proportional to system size. Large municipalities can afford full-time engineers. Smaller municipalities might use part-time or shared support. One expected function of a staff engineer should be process control decisions beyond the capacities of the on-site operators. A properly designed management organization would allow a single staff engineer to service several facilities in addition to providing technical assistance to other management levels. Other functions of technical staff would include participation and review of facility acquisitions, plant upgrade recommendations, technical coordination with regulatory authorities, monitoring construction and modifications, coordination for operator training and update, and interaction with consultants and outside advisors. Regardless of exact duties, however, the concept of the staff engineer as part of the management staff structure is to create the linkage between management and operations. This linkage is currently missing as part of the plant operator's man/organization interface.

It is the facility owner who must insure appropriate organizational considerations during plant design and operation. The owner has need of the facility, controls funds, contracts for plant design, and directs management of the new plant. No one else can be held solely responsible for proper management organization. The design engineer cannot be blamed for management deficiencies, because he contracts for plant design, not management organization design. The owner should solicit services of management and personnel administration specialists to provide proper organization design and construction. The owner should require that management organization designer and plant designer work together. Governmental agencies might play a role of identifying organization needs and making that information available to owners and facility management. Only through this kind of coordinated plant and organization design effort with explicit responsibilities can the management link to plant performance be assured.

IMPLICATIONS

Derivation of conclusions is inappropriate since the previous discussion was experiential and conceptual rather than experimental. Certain implications are appropriate, however. Application of Human Factors Engineering may be critical to the success or failure of wastewater reuse as a national policy. Acceptability of reuse appears to depend in large measure on the reliability, availability and maintainability of reuse facilities. Reliability, Availability, Maintainability (RAM) performance depends on both men and machines, and the man/machine interface is the critical linkage between the two. That interface is best designed using HFE concepts and techniques.

Researchers, developers and system designers may all have responsibilities for assuring component and system operability.

Best compromise probably exists between efforts to fit men to machines and machines to men. Such a compromise most likely depends on kinds and amounts of information derived from disciplines other than engineering. Reuse system design, therefore, may best be accomplished through use of interdisciplinary teams of people such as engineers, psychologists and personnel specialists.

Successful operation of advanced wastewater treatment (AWT) and reuse facilities may depend upon better information on operator job design and upon more accurate projections of future manpower resources than appear to be available at present.

Effective organization and management are essential to successful plant performance.

Organization design and building must commence simultaneously with plant design and building.

Professional staff must be made available to all levels of management, including the plant operator.

RECOMMENDATIONS

1. Studies are recommended to determine effective organization for treatment system management.

2. A series of continuing studies are recommended to determine who the treatment plant operator is and will be numerically, sociologically, economically, technically and psychologically.

3. Human Factors Engineering concepts and techniques are recommended for inclusion in graduate sanitary engineering curricula at the master's level to acquaint students with the need to consult and to utilize expertise outside the engineering professions.

4. Consideration should be given to imposing RAM specifications in addition to product quality specifications on new reuse facilities.

5. Studies are recommended to determine the proper roles of all personnel influencing treatment plant design and operation.

6. Sanitary engineering talent should be made available to all levels of management including the plant operator on a routine basis.

REFERENCES

1. Department of the Army Regulation AR 602-1, *Human Factors Engineering Program*. Washington, DC: Author unknown, 15 July 1976.

2. Gollan, Arye Z., Kenneth J. McNulty, Robert L. Goldsmith, Myles H. Kleper and Donald C. Grant. *Evaluation of Membrane Separation Processes, Carbon Absorption, and Ozonation for Treatment of MUST Hospital Wastes*. Final report on US Army Medical Research and Development Command contract DAMD 17-74-C-4066. Wilmington, MA: The Walden Research Division of Abcor, Inc., August, 1976.

3. Department of the Army. Request for Quotation DAMD 17-76-C-6561, *Fabrication of a Medical Unit, Self-contained, Transportable Water Processing Element Pilot Plant with Supporting Data Acquisition, Monitoring and Control System*. Washington, DC: US Army Medical Research and Development Command, 3 May 1976.

4. Lambert, Walter P. and LeRoy H. Reuter. ""Wastewater Reuse Within an Army Field Hospital." *Proceedings of the Third National Conference on Complete Water Reuse*. American Institute of Chemical Engineers and US Environmental Protection Agency Technology Transfer, 27-30 June 1976, pp. 447-456.

5. Yang, P.Y., W.P. Lambert, B.W. Peterman and J.D. Powell, Jr. "Instrumentation for Automating Water Processing Systems," Session 12, this Symposium.

6. Ongerth, H.J. and J. Crook, "Treatment Reliability and Effluent Quality Control for Potable Reuse. IN Linstedt, K. Daniel and Edwin R. Bennett (eds.), *Research needs for the Potable Reuse of Municipal Wastewater*. Cincinnati, OH: US Environmental Protection Agency, 1975, pp. 78-85.

7. US Environmental Protection Agency. *Technology Transfer*. Cincinnati, OH: Environmental Research Information Center, October, 1977.

8. Water Pollution Control Federation. *Highlights*, 14(7) 3, July, 1977.

9. Hegg, B.A., K.L. Rakness and J.R. Schultz, "Evaluation of Operation and Maintenance Factors Limiting Municipal Wastewater Treatment Plant Performance". *Journal of the Water Pollution Control Federation*, 50, 419(1978).

10. Robens of Woldingham, foreword to *The Human Operator in Process Control*, Elwyn Edwards and Frank P. Lees, editors. London: Taylor & Francis, Ltd., Halsted Press, 1974.

11. Luscombe, John S., "Computer-based Process Control." *J. AWWA*, 71(3)127-132, 1979.

12. Dillingham, James H.," Automating Distribution System Operations." *J. AWWA*, 71(3)133-135, 1979.

13. F. P. Lees, "Research on the Process Operator." IN *The Human Operator in Process Control*, Elwyn Edwards and Frank P. Lees, editors. London: Taylor & Francis, Ltd., Halsted Press, 1974, pg386.

14. "WPCF's Role in Solving O&M Problems, "*Deeds & Data*", June 1978.

15. Miller, E.J. and A.K. Rice, *Systems of Organization*. Tavistock Publications, New York, 1967.

16. McGregor, Douglas, *The Human Side of Enterprise*, McGraw-Hill Book Company, Inc., New York, 1960.

MULTIPURPOSE WATER TREATMENT SYSTEM DEVELOPMENT

M. K. Lee
Program Manager
Life Systems, Inc.
Cleveland, OH 44122

L. H. Reuter
Chief, Environmental Protection Research Division
U.S. Army Medical Bioengineering Research & Development Laboratory
Fort Detrick, Frederick, MD 21701

INTRODUCTION

Limited sources of natural fresh water, together with the possibility of ground water poisoning by the enemy, make the reclamation of wastewater for reuse extremely vital to the operation of the combat unit in water deficient areas. In response to this need, the U.S. Army Medical Research and Development Command has been developing a wastewater reuse system. The interim objective was reuse of nonsanitary wastewaters for nonpotable hospital requirements. The ultimate objective was reuse for potable and nonpotable requirements. A full-scale pilot plant of a water processing system, equipped with a data acquisition system, was designed and built by Life Systems, Inc. The Water Processing System treats nonsanitary wastewaters (kitchen, laundry, shower, hospital laboratory, operating room and X-ray wastewaters) for nonpotable reuse or for surface discharge to the environment and purifies natural fresh and brackish waters for potable use. The projected variations of the contaminant concentrations in the wastewaters are 50-1,000 mg/l total organic carbon and suspended solids, 300-6,000 mg/l chemical oxygen demand, 500-4,200 mg/l total solids and 5-900 Jtu for turbidity.

The Water Processing System pilot plant consists of four units: (1) a water treatment unit, (2) a water purification unit, (3) an ultraviolet/ozone oxidation unit and (4) an automatic instrumentation unit. In addition, the pilot plant is equipped with a data acquisition system. This paper describes in brief the design, configuration and operation of the Water Processing System.

The primary objective of the Water Treatment Unit is to remove suspended solids and turbidity from the wastewaters. When operated alone, the Water Treatment Unit treats the wastewaters for surface discharge. It consists of three unit processes: (1) equalization/prescreening, (2) ultrafiltration and (3) hypochlorination.

The major function of the Water Purification Unit is to remove most dissolved contaminants from process water streams. When operated alone, the Water Purification Unit purifies natural waters for potable use. It consists of three unit processes: (1) natural water pretreatment by the use of depth filtration, carbon adsorption and ion exchange, (2) reverse osmosis and (3) hypochlorination.

When both the Water Treatment Unit and the Water Purification Unit are operated in series, the Water Processing System produces nonpotable reuse water from the wastewaters. The Ultraviolet/Ozone Oxidation Unit is a supplementary unit to be used with the Water Treatment Unit or the Water Purification Unit while treating certain types of wastewaters with high organic concentrations. The ultraviolet/ozone oxidation is used as a final purification step to destroy and eliminate residual organic contaminants in the process water. The Automatic Instrumentation Unit provides the control and monitor functions for the above three units.

The Water Processing System has a nominal product water capacity of 3,500 gal per 20-hr day. The product water recovery is at least 85% of inflow in the production of reuse water and 90% in the production of potable water. The overall contaminant removal efficiencies are approximately 98.9% for total organic carbon, 99.5% for chemical oxygen demand and 98.5% for total solids.

A sufficient, reliable supply of safe drinking water has played one of the most important roles on fighting in water-deficient areas such as Africa and Indochina during World War II, Vietnam and Middle East Asia. Recently, a safe water supply has been recognized as important to the success of the U.S. Army operation as are other expendable supplies such as ammunition and food(1). Unlike ammunition and food, safe drinking water can be produced on-site near the battle field and wastewaters generated in the Army installations can be recycled for reuse after treatment. Such water management may significantly reduce or ultimately eliminate the logistical burden of the Army operation associated with hauling a huge amount of water to each new field installation. In addition, American soldiers' health and lives are not permitted to be in danger as a result of drinking natural waters poisoned by the enemy.

Highlights and Benefits of the Water Processing System

The Water Processing System (WPS) pilot plant is one of the most advanced water treatment systems in which a number of state-of-the-art technologies and more recent advances in water processing have been integrated into a compact design. It has most of the features of the prototype WPS which can be transported to a point of mission via conventional routes such as standard cargo trucks, external helicopter loads, railroad, ship or cargo aircraft. In addition to treatments of eleven different types of wastewaters considered for design, the pilot plant also can be used as a test bed for the general purpose of water treatment. Major features of the WPS are summarized in Table 1.

The unit processes employed in the WPS include three recent advances in water treatment: ultraviolet/ozone oxidation (O_3/UV), reverse osmosis (RO) membrane separation and ultrafiltration (UF) membrane separation. The O_3/UV is a chemical treatment process in which organic impurities are oxidized by ozone and UV light to carbon dioxide, water and other simple, low molecular weight compounds. It also can provide a 100% killing of bacteria, viruses or other known microorganisms, a complete destruction of color and odor, and an improvement of taste. Both the RO and the UF separation techniques are physical treatment processes in which a semipermeable membrane separates dissolved or suspended contaminants from permeable water. The overall efficiency of the WPS for removal of various contaminants is approximately 99%.

The pilot plant employs an advanced instrumentation concept which is highlighted by minicomputer-based automatic control and monitor and by the capability of the fault detection/isolation and performance trend analysis. A number of flexibilities such as semiautomatic instrumentation and manual overrides for major components are incorporated for pilot plant testing. The pilot plant can be operated at a remote terminal with all of its control, monitor and data acquisition benefits.

The unique requirements for the WPS design were:

1. Limited allowance on dimensions, weight and power consumption for transportation and field applications.
2. Automatic instrumentation and minimum maintenance for unskilled operators.
3. Pilot plant capability and semiautomatic instrumentation for performance evaluation and engineering data development.

SYSTEM DESCRIPTIONS

The WPS is an integrated multipurpose water treatment system which fulfills three functional requirements: (1) potable water production from natural waters, (2) nonpotable reuse water production from various types of wastewaters and (3) treatment of the wastewaters for safe discharge to the environment.

Simplified Flow Diagram

Figure 1 is a simplified flow diagram of the WPS. The wastewaters collected in the equalization tank are pumped through UF membranes. Depending on the mode of operation, the permeate stream of the UF membranes is further purified for reuse by RO membranes, or discharged to the environment either through a hypochlorination (HC) unit (calcium hypochlorite mixers) or through an ozone contactor and the HC unit. The permeate stream of the RO membranes flows to the reuse water storage tank either through the other HC unit (lower one) or through the ozone contactor and the HC unit, depending on the organic content in the wastewater. Ozone is generated from oxygen-in-air. Potable water is produced by purifying natural fresh or brackish water. The natural water purification follows a sequence of depth filtration (DF), carbon adsorption (CA), ion exchange (IE), RO and HC processes. More detailed descriptions and operation of the WPS are presented in the following sections.

Wastewater Sources and Product Waters

The WPS pilot plant was designed to treat two types of contaminated water: natural waters and nonsanitary wastewaters generated in the Field Army Medical Facilities. In addition, the pilot plant may be used to conduct treatability studies for various types of wastewaters generated by industries and other fixed installations.

Natural waters are further classified as fresh and brackish. The quality of the natural waters highly depends on the geographic location, industrial pollution and the activities of man and animal. The compositions and general characteristics of the brackish waters, used as a guideline for the WPS design, are shown in Table 2. The values were mostly based on the field analysis data(3) of three surface water samples (Buckeye, Arizona; Denver, Colorado; Roger Springs, Nevada). The quality of the fresh waters is expected to be much better than that of the brackish waters.

The nonsanitary wastewaters considered for the pilot plant design were: laundry (Type I - colorfast, Type II - woolens, and their mixtures), shower and kitchen wastes; hospital laboratory, X-ray and operating room wastes generated in the U.S. Army field medical facilities; and a composite waste (defined in Table 4). The overall wastewater characteristics are presented in Table 3. Detailed simulation and compositions of the various wastewaters were published elsewhere(1,2,4).

Based on the wastewater characteristics, and for convenience in the process control, the wastewaters were classified into four groups. Table 4 shows the classification of wastewater sources, along with the product waters. The product waters are classified into three groups according to the purpose of water treatment: (1) discharge water, (2) nonpotable reuse water and (3) potable water. Each classification of the wastewater sources requires a different combination of treatment processes.

Mode of Operation

The WPS can be operated in either one of two operational modes: Reuse and Potable/Discharge (Table 5). In the Reuse Mode the WPS treats and purifies nonsanitary wastewaters for nonpotable reuse. In the Potable/Discharge Mode it simultaneously treats those same wastewaters for discharge to the environment while treating natural fresh or brackish water for potable use. The overall recovery of the product water is at least 85% of inflow in the Reuse Mode and 90-95% in the Potable/Discharge Mode.

Reuse Mode. Figure 2 is a block diagram of the Reuse Mode. The wastewaters (wastewater Sources C and D) are fed to the equalization/prescreening (EP) process in which gross suspended solids are removed. In the UF process, suspended solids and dissolved solutes with a molecular weight greater than approximately 15,000 are separated to minimize the fouling and maintenance of the RO membranes. The function of the RO process is to remove most of the dissolved organics with a molecular weight of 150 to 15,000. The residual low molecular weight organic solutes remaining in the RO permeate of the Source D wastewater are finally oxidized in the O_3/UV process to meet the water quality specifications(4) for reuse water: 5 mg/l total organic carbon (TOC) and 10 mg/l chemical oxygen demand (COD), or less. The O_3/UV process is not needed for the treatment of the wastewater Source C. The HC process is used to maintain 5 mg/l free residual chlorine in the product reuse water.

The typical distribution of TOC, COD and total solids concentrations for the composite wastewater is shown in Figure 2. The numbers in parentheses are the typical rejection percents for each unit process. The overall contaminant removal rates projected are 98.9% for TOC, 99.5% for COD and 98.5% for total solids.

Potable/Discharge Mode. Figure 3 is a block diagram of unit processes employed in the Potable/Discharge Mode. The WPS performs two separate, independent functions simultaneously: (1) potable water production from natural fresh or brackish water and (2) wastewater treatment to protect the environment from toxic waste discharge. A total of nine processes were included in both treatment trains. The RO process is the main stage of the potable water treatment, while the UF process is the heart of the wastewater treatment for discharge. Each train has its own HC unit. The O_3/UV process in the surface discharge train is used only for the wastewater Source B with a high organic loading. The primary objective of the natural water pretreatment is to minimize the fouling and maintenance of the RO membranes by removing suspended solids, minerals and organics.

System Design and Integration

For ease of handling and transportation the nine treatment processes employed in the WPS pilot plant were integrated into three units: a Water Treatment Unit (WTU), a Water Purification Unit (WPU) and a O_3/UV Unit. The WTU consists of EP, UF and HC processes. The WPU contains the processes of DF, CA, IE, RO and HC. The physical dimensions (length x width x height) of the three units are as follows:

WTU: 12 x 8.75 x 6.75 ft
WPU: 9.75 x 5 x 6.75 ft
O_3/UV: 10 x 8.5 x 6.75 ft

The control and monitor instrumentation for the three units is provided by an Automatic Instrumentation Unit which has been separately packaged into a 21 x 21 x 28.5 in. enclosure. The brief description of the four units are presented in the following subsections.

Water Treatment Unit. The simplified flow schematic of the WTU is shown in Figure 4. The wastewaters are fed to the EP tank where hydraulic loading and concentration variations are equalized to result in a more uniform feed to the UF membranes. The wastewaters are then pumped through a 40-mesh basket strainer and a heat exchanger to the UF membrane assembly. The permeate stream of the UF assembly is finally fed to the WPU in the Reuse mode, or to the O_3/UV Unit for wastewater Source B (see Table 4) in the Discharge mode, or to the HC unit for wastewater Source A in the Discharge mode.

The EP tank fabricated of fiberglass has a wet volume of 1,320 gal and is divided into four compartments: a bad actor tank (224 gal); a mixing zone (134 gal); a clarifier section (883 gal) and a UF feed tank (80 gal).

The UF feed pump feeds the wastewater to the UF circulation loop at a rate of approximately 20 gpm. The high flow pump feeds both the recycle and feed water to the UF membranes at a rate of 270 gpm. The UF membrane assembly consists of nine modules which are installed in parallel. Each module has eight UF membranes in series. The membrane element (Abcor HFD-251G) is 1-in. diameter x 10 ft long with an active surface area of 2.2 sq ft. The total surface area of the 72 membranes installed in the UF assembly is 158 sq ft. The flow rate of the UF permeate varies with the operating conditions and the wastewater characteristics. Only the wastewater temperature is controlled by a heat exchanger to maintain the permeate flow above the required minimum of 3.26 gpm. A small portion of the recycle flow is bled to the EP tank to maintain a constant recycle flow rate. The UF process was designed for a 95% water recovery.

The function of the HC unit is to maintain 2 mg/l free-residual chlorine in the discharge water for disinfection. In order to maintain the required chlorine level, a 7% calcium hypochlorite solution, stored in a 50 gal polyethylene tank, is metered by a metering pump and mixed with the UF permeate in static mixers. The flow rate of the hypochlorite is automatically controlled in a feed forward mode by means of an upstream flow sensor and pump control logic.

The front and rear views of the WTU are shown in Figures 5 and 6, respectively. Figure 5 shows the UF membrane assembly, a high flow pump for circulation of UF recycle and feed flow, a sludge pump for removal of settled material in the EP tank, a HC unit and a semiautomatic instrumentation panel. The semiautomatic instrumentation is highlighted by five automatic fail-safe shutdown controls, thirteen digital readout monitors, ten warning and alarm lights, and 26 controls for pumps and valves.

Water Purification Unit. The flow schematic of the WPU is shown in Figure 7. Depending on the mode of operation, there are two streams of feed water to the RO feed tank: (1) the wastewaters treated in the WTU (Reuse Mode) and (2) natural waters being treated in a depth filter, a carbon column and an IE resin column (Potable/Discharge Mode). The water collected in the RO feed tank is then pumped through two heat exchangers and a series of 5- and 1-micron filters to the RO membrane assembly. The concentrate stream of the RO membrane assembly is recycled through a backpressure regulator and static mixers to the feed tank. A portion of the concentrate stream is bled to an incinerator in order to control the concentration buildup of contaminants in the system. The permeate stream of the RO membrane assembly is fed either to the O_3/UV Unit in the Reuse Mode or to the HC unit in the Potable/Discharge Mode. The HC unit is identical to that used in the WTU and is used to maintain 5 mg/l free-residual chlorine in the potable and reuse waters.

Natural water pretreatment consists of DF, CA and IE. Both the carbon column and the depth filter are 4.5 ft high with a 1-ft diameter and are constructed of carbon steel with an epoxy lining. The carbon column contains 2 cu ft of granular activated carbon (Calgon Filtrasorb 200). The depth filter consists of three beds: anthracite (20-in. bed height), sand (10 in.), and gravel (6 in.). The 3-ft diameter x 5-ft high IE column contains 21 cu ft of resin (Rohm & Haas Amberlite 200). The filtration and adsorption columns were designed to allow at least 50% bed expansion during backflushing. The ion exchange resin is regenerated by passing a 10% NaCl solution through the resin bed for 30 min. Approximately 100 gal of a concentrated NaCl solution are stored in a 210 gal brine tank. The regeneration of the ion exchange bed and the backflushing of the filtration and adsorption beds are automatically accomplished when the IONEXCH REGEN button on the WPS control panel is depressed.

The RO process is the heart of the WPU. The process water is pumped at the rate of approximately 6.6 gpm into the RO membrane assembly. The flow rate of the RO permeate varies with time, the operating conditions and the wastewater characteristics. Typical variations of the permeate flow rate range from 3.3 to 4.5 gpm. The flow rate of the bleed stream is set at 0.33 gpm. The backpressure regulator installed in the concentrate stream of the RO membrane assembly maintains the operating pressure at 815 psia. The RO process was designed for a 90% water recovery.

The front and rear views of the WPU are shown in Figures 8 and 9, respectively. Figure 8 shows the RO membrane assembly, an IE brine tank, an IE resin column and semiautomatic instrumentation panels. The RO membrane assembly consists of four hollow fiber membrane elements (DuPont B-10 Permeator) installed in a three-stage configuration (2:1:1). The membrane element is 47 in. long with a 5.5-in. diameter and contains nearly a million hollow fibers(5). It has a total membrane surface area of about 1,800 sq ft. The semiautomatic instrumentation has seven automatic fail-safe shutdown controls, twelve readout monitors, ten warning and alarm lights and 29 controls for pumps and valves.

UV/Ozone Oxidation Unit. Figure 10 is a flow schematic of the O_3/UV unit process. The wastewaters treated either in the WPU (wastewater Source D) or in the WTU (wastewater Source B) are fed to the precontactor in which they are pretreated with the residual ozone contained in the off-gas of the main ozone contactor. The pretreated water is then pumped through a heat exchanger to the six-stage ozone contactor where the residual organic impurities are oxidized by ozone and UV light. The water flows in a co- and counter-current fashion through one stage after another. The product water is finally transferred to a HC unit. The ozone-air mixture from an ozone generator is fed to the spargers located at the bottom of the ozone contactor and vented through the precontactor to atmosphere.

The dimensions of the precontactor are 36 x 10 x 78 in. It has a wet volume of 100 gal to provide a 33-min space time for liquid. The liquid level sensors located in the precontactor maintain the water level at 63 to 66 in. The ozone contactor is composed of three reactor modules. Each module, consisting of two stages, is 60 x 10 x 78 in. and has 20 UV lamps. The nominal UV (253.7 nm) output of a single lamp is rated at 26.7 W. Electric power for the UV lamps is provided by 30 ballasts. Each contactor stage has its own sparger which is fabricated of porous sintered stainless steel plate with a 5-micron pore size.

The wet volume of the ozone contactor was sized by using the following reaction kinetic equation(6):

$$-dC_B/dt = k_o \bar{P}_{oz}^{1.5} V_s^{1.5} C_B \tag{1}$$

where C_B = TOC concentration,
t = time,
k_o = rate constant,
$\bar{P}_{oz}$ = log mean of ozone partial pressure in the reactor, and
V_s = air superficial velocity.

Since water flow through the reactor closely approximates the ideal backmix flow(7), multiple stages were needed to reduce the reactor volume. Based on four stages and the backmix flow design equation, a total wet volume of 363 gal was specified in order to reduce the TOC concentration of the composite RO permeate from 30 mg/l to less than 5 mg/l. Two more identical stages (one module) were added to provide the pilot plant with the capability of treating other wastewaters than those wastes considered here. The resulting total wet volume of 544 gal for six stages provides a space time of three hours.

The front and side view of the O_3/UV Unit are shown in Figures 11 and 12, respectively. Figure 11 shows an ozone generator, a semiautomatic instrumentation panel and an air compressor. The ozone generator (Orec Model O3D25-AR) produces 25 lb ozone/day (at 1% by weight) from air. The air compressor feeds 24 scfm of air at 60 psig to the ozone generator. The semiautomatic instrumentation includes seven automatic fail-safe shutdown controls, eight digital readout monitors, ten warning and alarm lights and 26 controls for pumps, valves and UV lamps. Figure 12 shows the ozone contactor, UV lamps, an ultrasound generator and the air compressor. The ultrasound generator is incorporated into the first stage to enhance the ozone mass transfer rate.

Automatic Instrumentation Unit. The complexity of the WPS automatic instrumentation resulted from the requirements to: (1) control flow paths through various combinations of unit processes depending on the wastewater source; (2) control various process parameters at different setpoints depending on the mode of operation; (3) monitor the system performance for the performance trend analysis and for fault detection, isolation and correction instructions and (4) sample the system performance data for the Data Acquisition System (DAS). Figure 13 shows the hardware block diagram of the Automatic Instrumentation Unit. A minicomputer with a 16K, 16-bit word memory capacity was used as a central processor of the Automatic Instrumentation Unit to control six unit processes and provide communications with the operator and the DAS.

Figures 14 and 15 show the front and rear panels of the Automatic Instrumentation Unit. The front panel consists of a system status summary, a message display screen, a control panel, a keyboard panel and a recessed manual override panel.

The system status summary has four lights: NORMAL, CAUTION, WARNING and ALARM. Except for the NORMAL status, messages are displayed on a cathode-ray tube (CRT) screen indicating the cause of the CAUTION, WARNING or ALARM. The control panel provides the operator with pushbutton switches and lamp indicators for ease of selecting a system command (POWER, NORMAL, STANDBY or SHUTDOWN), wastewater sources and product waters, and auxiliary maintenance controls such as UF membrane cleaning, RO membrane cleaning, IE bed regeneration and system drain.

The keyboard panel and the message display screen provide the operator with a convenient way of communicating with the system. Through the keyboard panel the operator can examine and modify control or monitor setpoints of various process parameters. On-line display of parametric data also can be requested. The requested information as well as operator error messages and other messages of fault and performance trend analyses are displayed on the CRT screen. Manual override switches on the recessed panel are used to manually control 39 major components of the WPS, such as pumps and valves, for maintenance and the flexible pilot plant operation.

There are two communication links between the Automatic Instrumentation Unit and the DAS. One link is through the distributed input/output channel to the DAS foreground computer for on-line data communication and the other is through the communication switch of the DAS background computer to remote terminals for troubleshooting and debugging. The analog and digital interface board shown in Figure 15 is designed to handle 64 analog sensors, eight analog actuators, 128 digital sensors and 64 digital actuators.

Operation and Maintenance

Operation. The WPS is protected from illegal operation by unauthorized personnel. A password is required to operate the control panel. The password is a valid four-digit code preprogrammed for the identification of the operator. Before attempting to operate the system all manual override switches should be placed in the AUTO position and the COMMAND ENABLED/DISABLED switch on the rear panel (Figure 15) should be set at the ENABLED position.

To start the operation, the valid password should be entered first through the keyboard. The start up and operation of the WPS is then accomplished by depressing the NORMAL button after the wastewater source is selected. Detailed descriptions and operations of the control switches, the keyboard panel and the message display screen can be found elsewhere(6)

The WPS is protected from damages which may result from any mistakes (unauthorized commands) of unskilled operators. The computer will help and lead the operator to correct his mistakes for a proper operation of the system.

The nominal operating conditions of the WPS are summarized in Table 6. The pressures of the feed waters to the UF and RO membranes are nominally 65 and 815 psia, respectively. The influent flow rates of the wastewaters, as well as their compositions, are highly variable with time. To maintain a reasonably constant process rate the EP tank should have 400 to 800 gal of reserve water to absorb both flow and concentration variations. The UV intensity of 1.89 W/l is defined as the total light output of UV lamps divided by the wet volume of the ozone contactor. The UV intensity, based on the actual UV output, is approximately 0.78 W/l. The WPS operates 20 hours/day.

Maintenance. The ultimate goal for the WPS development was to design a system essentially free of laborious and complicated manual maintenance which requires extensive knowledge and training. The WPS pilot plant design incorporated such features, wherever possible without too much sacrifice of the system weight and volume, by automating essential maintenance procedures. The only thing required for such automated maintenances is to press a button when the system notifies the operator of a need for maintenance. The WPS has four automatic controls for maintenance: (1) UF CLEAN for UF membrane cleaning, (2) RO CLEAN COAT for cleaning and coating of the RO membranes, (3) IONEXCH REGEN for regeneration of the IE resin and backflushing of both the depth filter and the carbon column and (4) SYSTEM DRAIN for draining all water tanks.

DESCRIPTION OF UNIT PROCESSES

A number of water treatment processes employed in the WPS can be grouped into six unit processes: (1) EP, (2) UF, (3) DF/IE, (4) RO, (5) O_3/UV and (6) HC. The selection of unit processes for treating specific wastewaters and the brief descriptions of the processes are presented in this section.

Selection of Unit Processes

The five classifications of the wastewater sources for the WPS were described in Table 4. Table 7 shows the selection of unit processes for treating each category of the wastewater sources. To eliminate the conflict of using the HC in the Discharge mode and in the Potable mode, two HC units were built in the WPS so both the Discharge and Potable modes can be operated at the same time. Hence, the WPS operates in either one of two modes: Reuse or Potable/Discharge.

Process Descriptions

Equalization and Prescreening. The function of the EP process is to (1) remove, by settling and screening, all large influent wastewater solids (i.e., bandages, organs, bones, fragments, paper, waste food, etc.), (2) equalize the hydraulic loading variations to result in a constant flow to the UF unit process and (3) dampen the refractory organics and biological loading fluctuations to result in a more uniform feed to the UF process.

All wastewaters, except laboratory waste, flow directly into the mixing compartment of the EP tank. Since laboratory waste is high in organic concentration it is first collected in the bad actor tank. The laboratory waste is allowed to flow into the mixing compartment only when the liquid level in the bad actor tank is high. This has a dampening effect on the organic concentration fluctuation. The pH adjustment in the mixing compartment induces precipitation of some dissolved solutes and increases the contaminant removal efficiencies of the UF and RO membranes. Sludge accumulated at the bottom of the tank is pumped to an incinerator once a day.

Ultrafiltration. Ultrafiltration is a membrane separation process in which a semipermeable membrane separates suspended solids and some dissolved solutes from permeable solvent. The separation occurs either by the sieving mechanism or due to the difference in the mass transfer rates through the membrane. Ultrafiltration is carried out at reasonably low pressures, typically less than 150 psia.

The major function of the UF process in the WPS is to remove suspended solids and turbidity from the wastewaters in order to minimize the fouling and maintenance of the RO membranes. Due to the high contaminant concentrations in the wastewater, the permeate flux of the UF membrane is relatively low. Thus the flux is of prime concern in order to reduce volume, weight and power consumption of the WTU.

As UF processing continues, the permeate flux declines due to the solid concentration buildup in the feed and the increased concentration polarization on the membrane surface. Under such circumstances the gel-layer mass transfer rather than the membrane permeability controls the flux (the gel-layer is a layer of highly concentrated, precipitated solids which is formed on the active surface of the UF membrane). The permeate flux depends on both the flow velocity and the water temperature, since they play a significant role on the gel-layer mass transfer. The UF membranes in the WPS operate in the gel-layer mass transfer-controlled regime. The membrane permeate flux increases significantly as the temperature of the process water or the flow velocity increases. Operating at too high temperatures, however, should be avoided due to disadvantages such as: greater power required for heating in the UF process and for cooling in the RO process, insulation needed for the EP tank, and more costly high temperature resistant material. For the operation of a high water recovery, the concentrate stream of the UF membranes usually must be recycled to maintain a desired flux.

The contaminant removal efficiency, R_m, of a membrane may be defined as:

$$R_m = 1-C_p/C_f \qquad (2)$$

where C_p and C_f are the contaminant concentrations in the permeate stream and the feed water, respectively.

A simplified model was developed to characterize the performance of the UF process. Overall material balances for the semibatch operation (one batch a day) were used to predict the system removal efficiencies and the concentration variation of a certain contaminant in the system. The results are as follows:

Contaminant	Membrane Rejection, R_m	Process Efficiency, R_o	Concentration Ratio, C_s/C_i
Suspended Solids	0.999	0.997	19.9
Turbidity	0.996	0.989	19.8
Total Solids	0.50	0.26	6.1
TOC	0.88	0.73	14.8
COD	0.90	0.76	15.6

The contaminant removal efficiencies, R_o, of the UF process vary depending on the membrane rejection and the amount of water initially charged to the EP tank. The removal efficiencies increase with the decrease of the initial charge. On the other hand, the permeate flux decreases, since the contaminant concentration in the system builds up more rapidly. An initial charge of 412 gal of water was used for the calculations. The last column gives the ratio of the contaminant concentration in the system to that of the influent water flowing in the EP tank.

Depth Filtration and Ion Exchange. The DF/IE is a pretreatment process for purifying natural waters. The natural water pretreatment was needed to minimize the fouling and maintenance of the RO membranes. Among various foulants of the RO membranes, turbidity, suspended solids and hardness are of prime concern in the WPS operation. Suspended solids should be reduced to less than 5 mg/l and the turbidity to less than 10 Jtu before processing by RO. Hardness producing ions such as calcium and magnesium should be removed sufficiently enough to prevent the precipitation of their sulfate or carbonate salts on the RO membrane surface. The primary function of the depth filter is to remove suspended solids and turbidity from the natural waters. The CA column was employed to remove organics which could foul the IE resin. The objective of the IE column was to remove multivalent metallic cations contributing to hardness.

The CA column in the WPU performs a dual function of filtering out turbidity and adsorbing dissolved organic materials, if any, which could be introduced into the natural waters from both natural and industrial sources. Such organic contaminants can cause taste, odor and color problems. Recent use of activiated carbon has been effective in drinking water treatments to eliminate or reduce such problems.

The IE process is illustrated in the following reversible reaction for cation exchange:

$$Ca^{++} + 2NaX = CaX_2 + 2Na^+ \qquad (3)$$

where X represents a stationary monovalent anionic site in the polyelectrolyte network of the cation exchange resin. Exchange of multivalent cations (Ca^{++}) in solution for monovalent cations (Na^+) in the resin phase occurs during the normal IE operation (forward reaction) because in dilute solutions a typical cation exchange resin has a greater affinity for cations having the higher charge density. After exhaustion of the resin exchange capacity, the resin should be regenerated according to the reverse reaction. During the regeneration step multivalent cations are removed from the resin by the use of a solution containing a higher concentration of monovalent cations, thus restoring the resin to its initial condition.

Reverse Osmosis. In the RO process, separation of dissolved organics and minerals usually occurs due to the difference in the mass transfer rates of water and solutes through a semipermeable membrane. Reverse osmosis is the reverse phenomenon of the natural forward osmosis. When two solutions of the same component with different concentraions are separated by a semipermeable membrane, the solvent in the dilute solution tends to flow through the membrane to the more concentrated solution, thereby diluting the latter. The solvent flow continues until pressures on both sides of the membrane are in equilibrium (osmotic equilibrium). In RO the direction of the natural osmotic flow is reversed by applying a hydraulic pressure to the more concentrated solution. To obtain reasonable flow rates of the solvent, the applied hydraulic pressure should be much greater than the osmotic pressure due to the concentration difference. The RO process is operated typically at 300 to 1500 psi differential.

Performance of a RO membrane is usually characterized by its solute rejection, as defined by equation 2, and by the permeate flux. Provided that transport of water through the membrane governs the rate of permeation, the permeate flux of water, J_p, can be related to the driving force, delta P, by

$$J_p = K \frac{(\Delta P)^n}{\mu \delta} \qquad (4)$$

where
μ = viscosity of water
δ = membrane thickness
K = permeability of the membrane, and
n = constant

The driving force, delta P, is defined by:

$$\Delta P = (p_c - p_p) - (\pi_c - \pi_p) \qquad (5)$$

where p is the hydraulic pressure, π is the osmotic pressure and the subscripts c and p signify the concentrate and the permeate stream, respectively. In a dilute, ideal solution, the osmotic pressure can be approximated as:

$$\pi \simeq cRT \qquad (6)$$

where
c = solute concentration,
R = gas constant, and
T = absolute temperature.

Modular membrane systems either for RO or UF separation can be designed to operate in one of several process configurations, i.e., semibatch, once-through continuous, recycle-and-bleed continuous, stages in series, stages in parallel, etc. Three common system designs and their descriptions were presented in a previous report.(9) In order to achieve a high product recovery (90%) and to maintain a desired flux, a recycle-and-bleed mode of the RO operation was selected in a staged arrangement of membranes.

Both the contaminant removal efficiency and the permeate flux of the RO membrane depend on the product water recovery. At a fixed product recovery the membrane permeate flux increases as the water temperature and the pressure differential across the membrane increase.

The contaminant removal efficiencies of the RO membrane used in the WPS are summarized below for both the composite waste and the natural brackish waters:(3,10)

- Composite Waste

Contaminant	Average Removal Efficiency, %
Total Solids	98
Conductivity	98
TOC	76
COD	76
Surfactants	97
Chloride Ion	98
Ammonia	47
Urea	86

- Natural Brackish Waters

Contaminant	Average Removal Efficiency, %
Total Solids	73 - 98
Chloride Ion	70 - 97
Nitrate-Nitrogen	65 - 85
Sulfate	88 - 99+

UV/Ozone Oxidation. The wastewaters contain a variety of toxic chemical compounds. The UF and RO processes are capable of reducing the concentration of many of these compounds to low levels (30 mg/l TOC and 108 mg/l COD for the composite). The processed reuse water must be essentially free of organics and inorganics which might be detrimental to man. A concentration level of 5 mg/l TOC and 10 mg/l COD has been suggested as an appropriate water quality specification(4). The O_3/UV has been demonstrated to be one of the most viable and cost-effective means of polishing the RO permeate to meet the required TOC and COD levels as established for the allowable concentration limits(3,11).

The main objective of the O_3/UV process in the Reuse Mode of the WPS operation is to reduce TOC from 30 mg/l to less than 5 mg/l (84% removal) and to reduce COD from 108 mg/l to less than 10 mg/l (91% removal). In addition, the O_3/UV process accomplishes disinfection (100% kill) and complete removal of color and foam.

The O_3/UV is a heterogeneous system in which ozone in the gas phase is contacted with organics in the liquid phase. At the ordinary test conditions organics may be assumed to be insoluble in the gas phase, hence, reactions occur in the liquid phase and ozone has to be dissolved in liquid to react with the organics.

When irradiated by UV light, dissolved ozone in the alkaline aqueous solution appears to decompose to hydroxyl radicals mostly by the following reactions.

$$O_3 + h\nu \rightarrow O_2^* + O^* \qquad (7)$$

$$O^* + H_2O \rightarrow 2OH^\cdot \qquad (8)$$

where * indicates an excited state. The hydroxyl radical ($OH^\cdot$) which has a higher oxidation potential than ozone is very reactive and seems to play a major role in the succeeding oxidations of organics. The hydroxyl radicals thus formed are then consumed on a competitive basis either for the oxidation of organics or for the radical-radical reactions.

The reaction mechanism for the organic oxidation is not known. It has been suggested that the hydroxyl radicals may undergo hydrogen abstraction reactions with aliphatic compounds. The resulting organic radical may react further to produce an alcohol:

$$RH + OH^{\cdot} \rightarrow R^{\cdot} + H_2O \qquad (9)$$

$$R^{\cdot} + OH^{\cdot} \rightarrow ROH \qquad (10)$$

In successive abstraction of hydrogens, addition of hydroxyl radicals or oxygen (which is present almost in any ozonation system) and rearrangement of unstable intermediates, alcohols may be oxidized through aldehydes to acids, as the experimental data by Chian and Kuo(12) for the methanol ozonation suggest. They identified formaldehyde (HCHO) and formic acid (HCOOH) as intermediate products. Further oxidation of acids would result in decarboxylation to produce CO_2 and water. The oxidation of methanol therefore may proceed as follows:

$$CH_3OH \xrightarrow[h\nu]{OH^{\cdot}} HCHO \xrightarrow[h\nu]{OH^{\cdot}} HCOOH \xrightarrow[h\nu]{OH^{\cdot}} CO_2 + H_2O \qquad (11)$$

Chemical oxygen demand is naturally reduced as the oxidation proceeds. However, no TOC reduction is realized unitl decarboxylation occurs.

Lee and Yang(13) identified the following variables as affecting the overall rate of the UV/ozone oxidation.

- TOC concentration
- Intensity of UV light
- Ozone concentration in feed gas
- Feed gas flow rate
- Temperature, pressure and pH

Their experimental results indicated that the overall reaction under the WPS design conditions proceeds through three phases with different rates. In phase II the reaction was first-order with respect to the TOC concentration and 1.5th-order with respect to the ozone partial pressure. It also had 1.5th-order dependence on the gas superficial velocity. The effect of UV light activation became more significant as the reaction proceeded. The effects of temperature, pressure and pH on the overall reaction rate were not significant under the operating conditions of the WPS O_3/UV Unit.

Hypochlorination. Hypochlorination in the WPS is the last process of a treatment train for any reuse, potable or discharge water production. In a treatment train including the O_3/UV process, almost complete destruction of bacteria, viruses or any other known microorganisms is accomplished. However, the product water of the O_3/UV process has a low level of dissolved ozone residual, typically less than 0.3 mg/l, and the dissolved ozone readily decomposes with a half-life less than one hour. It was expected that the dissolved ozone alone will do little to protect the product water from possible reinfection in the water distribution line.

The main objective of the HC unit in the WPU is to accomplish disinfection of the potable water and to provide a precautionary protection for possible reinfection of the reuse water in the distribution line. The function of the HC unit in the WTU includes disinfection of the discharge water which is not treated in the O_3/UV Unit. The HC is one of the conventional chlorination practices which uses a calcium hypochlorite solution instead of hazardous chlorine gas. The two HC units in the WPS provide a minium free-chlorine residual of 2 mg/l in the discharge water and 5 mg/l in the reuse of potable water. The hypochlorite dose rates are automatically controlled to maintain the desired chlorine levels.

SUMMARY

The following conclusions were drawn from this development program:

1. The integrated, multifunctional WPS is a viable solution to meet the needs for the multipurpose water treatments in the Field Army Medical Facilities.
2. The WPS pilot plant was designed to be capable of producing potentially nonpotable reuse water from nonsanitary wastewaters and producing potentially drinkable water from natural waters while treating the wastewaters for safe discharge to the environment. This is accomplished by the use of various combinations of the following six unit processes: EP, UF, RO, DF/IE, O_3/UV and HC.
3. A number of flexibilities incorporated into the WPS enable the pilot plant to serve as a test bed for treatment of other types of wastewaters.
4. Operation and maintenance of the WPS has been greatly simplified by the use of minicomputer-based automatic instrumentation. The complicated process and parameter controls for the multifunctional WPS are fully automated so that an operator with low skill levels and limited training can operate the system.
5. The WPS pilot plant is transportable via conventional routes.

ACKNOWLEDGMENTS

This program was supported by U.S. Army Medical Research and Development Command through Contract DAMD17-76-C-6063.

The authors wish to acknowledge the technical contributions by Dr. P. Y. Yang, D. C. Walter and G. G. See of Life Systems, Inc. and the program guidance and assistance by Major W. P. Lambert and W. J. Cooper of U.S. Army Medical Bioengineering Research and Development Laboratory, Fort Detrick, MD and by Dr. R. A. Wynveen of Life Systems, Inc.

REFERENCES

1. Lee, M. K.; Yang, P. Y.; Wynveen, R. A.; & See, G. G., "Pilot Plant Development of an Automated, Transportable Water Processing System for Field Army Medical Facilities," Final Report, Contract DAMD17-76-C-6063, ER-314-7-1, Life Systems, Inc. Cleveland, OH (Jun. 1978).

2. Lambert, W. P., & McCarthy, J. J., "Ozone Oxidation for Reuse of Army Field Hospital Wastewaters," IOI Forum on Ozone Disinfection, Ozone Press Int., 108 (1977).

3. Gollan, A. Z.; McNulty, K. J.; Goldsmith, R. L.; Kleper, M. H.; & Grant, D. C., "Evaluation of Membrane Separation Processes, Carbon Adsorption, and Ozonation for Treatment of MUST Hospital Wastes," Final Report, Contract DAMD17-74-C-4066, ADAO 30057, Walden Research Division of Abcor, Inc., Wilmington, MA (Aug. 1976).

4. "MUST Water Processing Element Pilot Plant Development," U.S. Army Medical Research and Development Command RFQ No. DAMD17-76-Q-6561 (May 3, 1976).

5. "Permasep Permeator Product Specifications," Dupont Company Technical Bulletin No. 260 (Mar. 1, 1976).

6. Lee, M. K.; See, G. G.; & Wynveen, R. A., "Reaction Kinetics of UV/Ozone with Organic Compounds in Hospital Wastewater," presented at the Symposium of Advanced Ozone Technology, sponsored by International Ozone Institute, Inc., Toronto, Canada, ER-345, Life Systems, Inc., Cleveland, OH (Nov. 16-18, 1977).

7. McCarthy, J. J.; Cowen, W. F.; Chain, E. S. K.; & Peterman, B. W., "Evaluation of an Air Stripping-Ozone Contactor System," Technical Report 7707, U.S. Army Medical Bioengineering Research and Development Laboratory, Fort Detrick, Frederick, MD (Jun. 1977).

8. Yang, P. Y.; Yeh, J. Y.; Powell, J. D.; & Wynveen, R. A., "Advanced Instrumentation Development for a Water Processing Pilot Plant for Field Army Medical Facilities," Technical Report, Contract DAMD17-76-C-6063, ER-314-7-4, Life Systems, Inc., Cleveland, OH (May 1978).

9. Lee, M. K.; Yang, P. Y.; & Wynveen, R. A., "Water Treatment Unit Development for Field Army Medical Facilities," Technical Report, Contract DAMD17-76-C-6063, ER-314-7-2, Life Systems, Inc., Cleveland, OH (Apr. 1978).

10. Lee, M. K.; Yang, P. Y.; & Wynveen, R. A., "Water Purification Unit Development for Field Army Medical Facilities," Technical Report, Contract DAMD17-76-C-6063, ER-314-7-3, Life Systems, Inc., Cleveland, OH (Apr. 1978).

11. See, G. G.; Yang, P. Y.; & Kacholia, K. K., "Research and Development of an Ozone Contactor System," Final Report, Contract DAMD17-76-C-6041, ER-300-4, Life Systems, Inc., Cleveland, OH (Jun. 1976).

12. Chian, E. S. K. & Kuo, P. P. K., "Fundamental Study on the Post Treatment of RO Permeates from Army Wastewaters," Second Annual Summary Report, Contract DAMD17-75-C-5006, UILU-ENG-76-2019, University of Illinois, Urbana, IL (Oct. 1976).

13. Lee, M. K. & Yang, P. Y., "UV/Ozone Oxidation Technology Development for Water Treatment for Field Army Medical Facilities," Technical Report, Contract DAMD17-76-C-6063, ER-314-7-5, Life Systems, Inc., Cleveland, OH (Jul. 1978).

Table 1
Major Features of the Water Processing System

- Advanced Water Treatment Processes
  - UV/Ozone Oxidation
  - Reverse Osmosis Membrane Separation
  - Ultrafiltration Membrane Separation
- Minicomputer-based Automatic Instrumentation
- Single Button Startup
- Automatic, Fail-Safe Shutdown Controls in Case of Emergency
- Unattended Operation
- Remote Operation and Monitoring
- Written Communication Between Operator and the System
- Fault Detection and Isolation Analysis
- System Performance Trend Analysis
- Self-Protection from the Damages due to the Operator's Error
- Only Authorized Personnel with a Valid Password Can Operate the System
- Pilot Plant Flexibilities for Scientific and Engineering Data Generation
- Interface with an Automatic Data Acquisition System

Table 2
Anticipated Water Quality for Brackish Surface Waters

Contaminants	Concentration, mg/l
General	
Total Dissolved Solids, mg/l	1,800-3,200
Suspended Solids, mg/l	10-100
Turbidity, Jtu	5-200
Conductivity, µmhos/cm	2,000-9,000
pH-Value	6-9
Hardness (as $CaCO_3$), mg/l	200-2,000
Metals	
Calcium, mg/l	50-500
Copper, mg/l	0-2.0
Iron, mg/l	≤ 1
Magnesium, mg/l	20-200
Sodium, mg/l	300-1,200
Potassium, mg/l	1-30
Manganese, mg/l	0-1.5
Nonmetals	
Bicarbonate or Carbonate, mg/l	100-500
Chloride, mg/l	300-1,500
Nitrate-Nitrogen, mg/l	5-50
Total Phosphate, mg/l	0-20
Sulfate, mg/l	100-1,800
Bromide, mg/l	0-10
Fluoride, mg/l	0-10
Miscellaneous	
Silica, mg/l	5-50
Hydrogen Sulfide, mg/l	Trace

Table 3
Overall Wastewater Characteristics

Contaminants	Range of Concentration Variation
Total Organic Carbon (TOC), mg/l	50 - 1,000
Chemical Oxygen Demand (COD), mg/l	300 - 6,000
Suspended Solids, mg/l	50 - 1,000
Total Solids, mg/l	500 - 4,200
Turbidity, Jtu	5 - 900

Table 4
Wastewater Sources and Product Waters

Wastewater Source		Product Water
Classification	Source	
A	Shower[a], Operating Room, Laundry[b]	Discharge
B	Kitchen, Lab, X-ray, Composite[c], Other Unknown Wastes	Discharge
C	Kitchen, Shower	Reuse
D	Laundry, Operating Room, Lab, X-ray, Composite, Other Unknown Wastes	Reuse
E	Natural Fresh Water, Natural Brackish Surface Water	Potable

---

(a) Shower waste includes lavatory water.

(b) Laundry wastes include Type I (color-fast), Type II (woolens) and their mixtures.

(c) Composite waste is defined as a mixture of shower (51%), kitchen (12%), laboratory (8%), X-ray (3%), and operating room waste (26%).

Table 5
Operational Modes of the Water Processing System

Operational Mode	Function	Product Recovery
1. Reuse	• Treatment and Recycle of Nonsanitary Wastewaters	85%
2. Potable/ Discharge	• Treatment of Natural Fresh and Brackish Waters for Potable Use	90%
	• Treatment and Discharge to the Environment of Nonsanitary Wastewaters	95%

Table 6
Nominal Operating Conditions of the Water Processing System

Operating Temperature, F	68 to 125
Operating Pressure, psia	
• UF Membrane	65
• RO Membrane	815
• Ozone Contactor	20
Water Flow Rates, gpm	
• Influent	
- Hospital Wastewaters	3.43
- Natural Waters	3.26
• Product	2.93
• UF Feed	20
• UF Concentrate Recycle Flow	250
• RO Feed	6.6
• RO Concentrate Recycle Flow	2.47
• RO Bleed	0.32
Ozone Contactor	
• Ozone/Air Flow Rate, scfm (at 70 F, 1 atm)	24
• Ozone Concentration, wt %	1
• UV Intensity Based on the Total Lamp Output, W/l	1.89
• UV Intensity Based on the UV Output, W/l	0.78
• Initial pH of Water	9
Free Available Chlorine Residual, mg/l	
• Potable and Reuse Water	5
• Discharge Water	2
Sludge Removal from EP Tank, gpd	132
Initial Charge to EP Tank, gal	400 to 800
Operating Time, hr/day	20

Table 7
Selection of Unit Processes

	Wastewater Sources(a)				
Unit Process	A	B	C	D	E
1. Equalization/ Prescreening	(1)	(1)	(1)	(1)	
2. Ultrafiltration	(2)	(2)	(2)	(2)	
3. Depth Filtration/ Ion Exchange					(1)
4. Reverse Osmosis			(3)	(3)	(3)
5. UV/Ozone Oxidation		(3)		(4)	
6. Hypochlorination	(3)	(4)	(4)	(5)	(3)
Product Water	Discharge		Reuse		Potable

(a) Wastewater Sources -
A. Shower, Laundry, Operating Room.
B. Kitchen, Laboratory, X-ray, Composite, Unknown
C. Kitchen, Shower
D. Laundry, Operating Room, Lab, X-ray, Composite, Unknown
E. Natural Fresh, Brackish

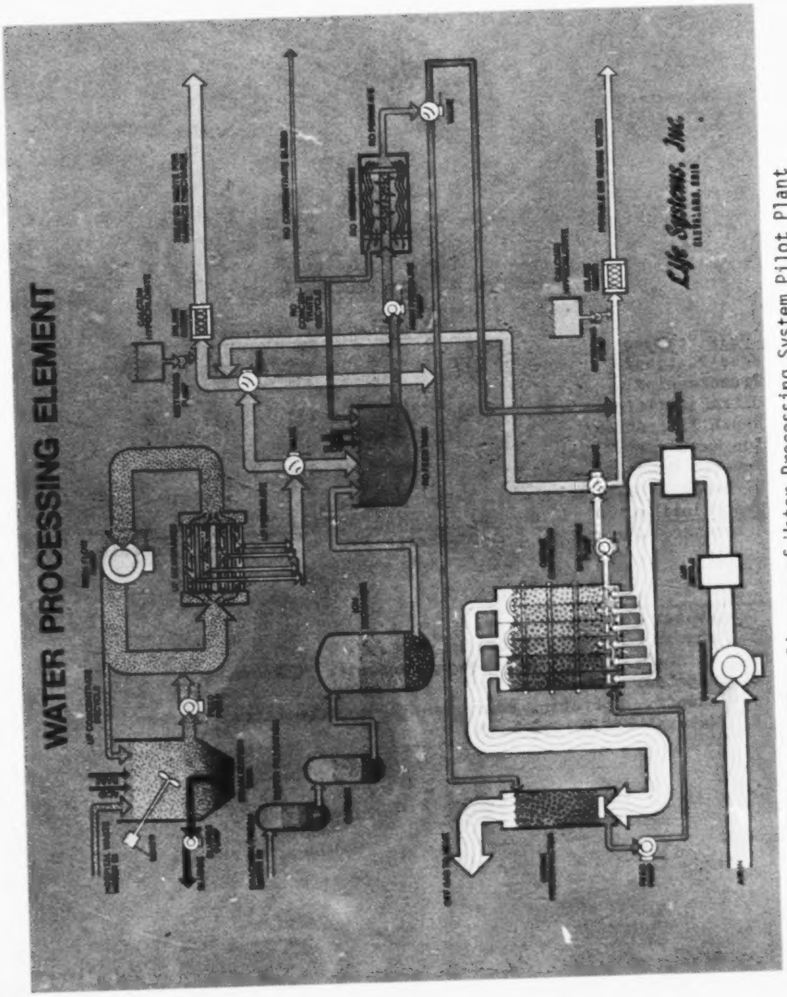
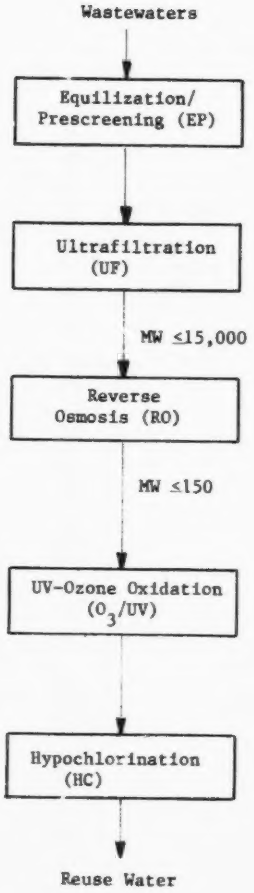
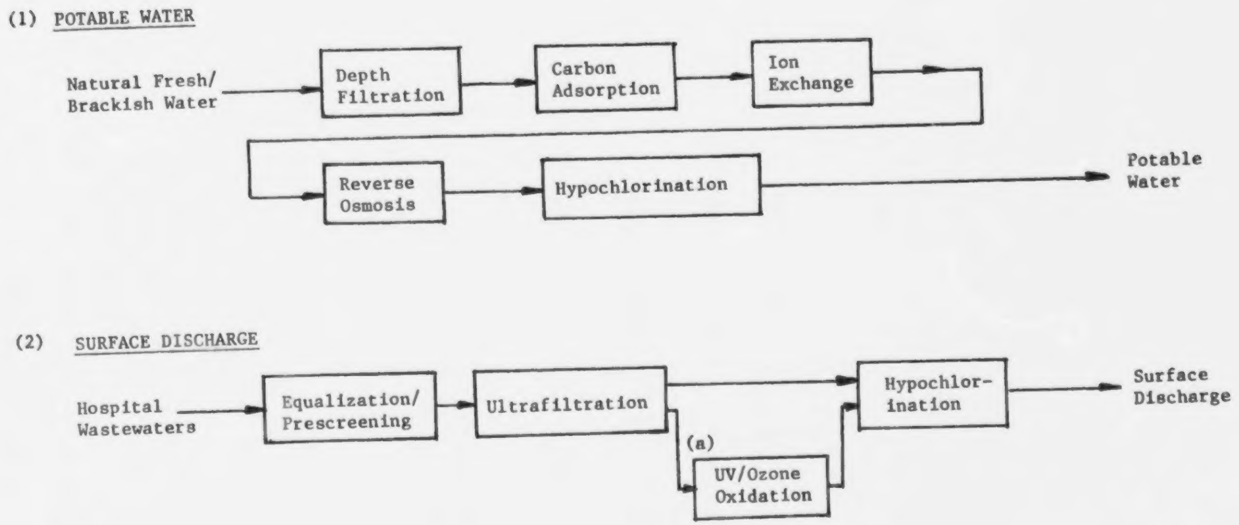


Figure 1 Flow Diagram of Water Processing System Pilot Plant



	TOC (mg/1)	COD (mg/1)	Total Solids (mg/1)
Wastewaters	50-1,000	300-6000	500-4,200
Equilization/ Prescreening (EP)			
	463	1,875	1,240
Ultrafiltration (UF)	(73%)	(76%)	(26%)
MW ≤15,000	124	450	918
Reverse Osmosis (RO)	(76%)	(76%)	(98%)
MW ≤150	30	108	18
UV-Ozone Oxidation (O_3/UV)	(≥84%)	(≥91%)	--
	≤5	≤10	--
Hypochlorination (HC)	--	--	--
Reuse Water	≤5	≤10	--
Total Removal %	(≥98.9)	(≥99.5)	(≥98.5)

Figure 2 Unit Processes Involved in Reuse Mode



(a) For wastes with high organic loading such as kitchen, lab, X-ray and composite.

Figure 3 Unit Processes Involved in Potable/Discharge Mode

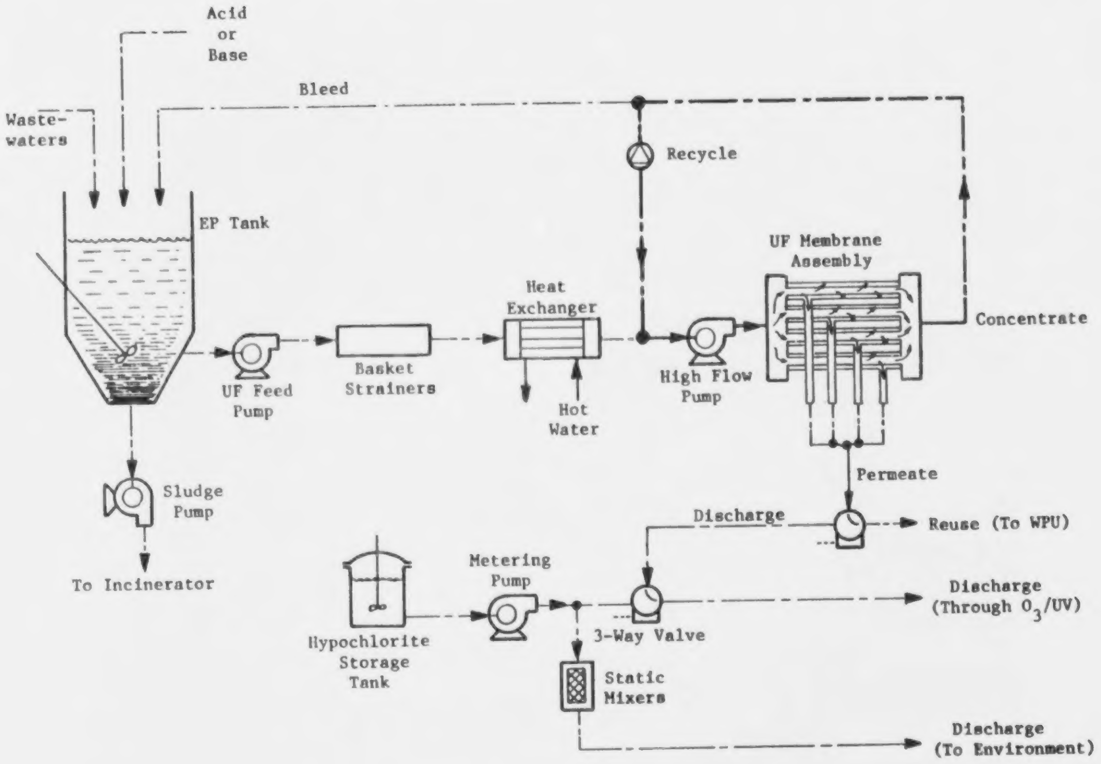


Figure 4 Flow Schematic of Water Treatment Unit

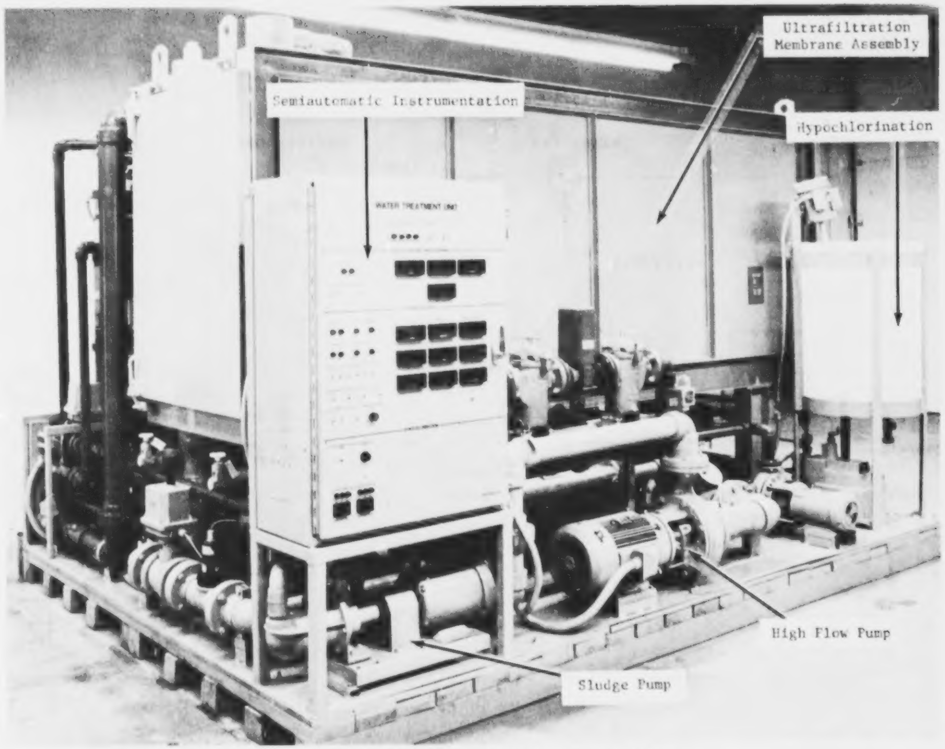

Ultrafiltration Membrane Assembly
Semiautomatic Instrumentation
Hypochlorination
WATER TREATMENT UNIT
High Flow Pump
Sludge Pump

Figure 5 Water Treatment Unit. Front View

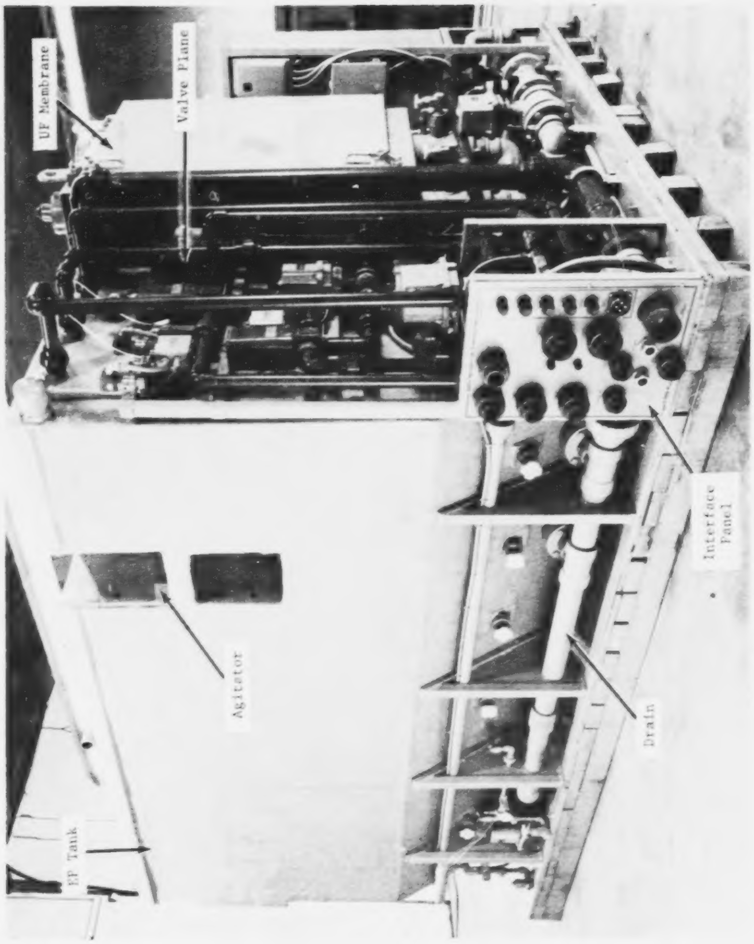


Figure 6 Water Treatment Unit, Rear View

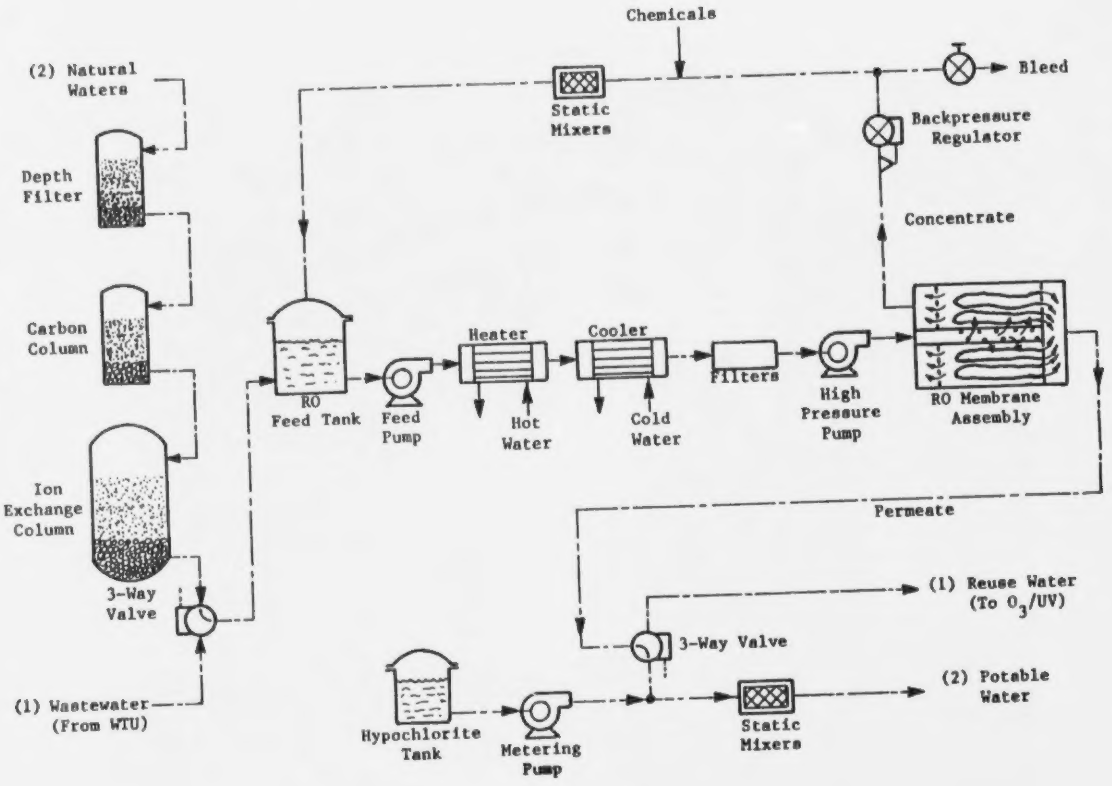


Figure 7 Flow Schematic of Water Purification Unit

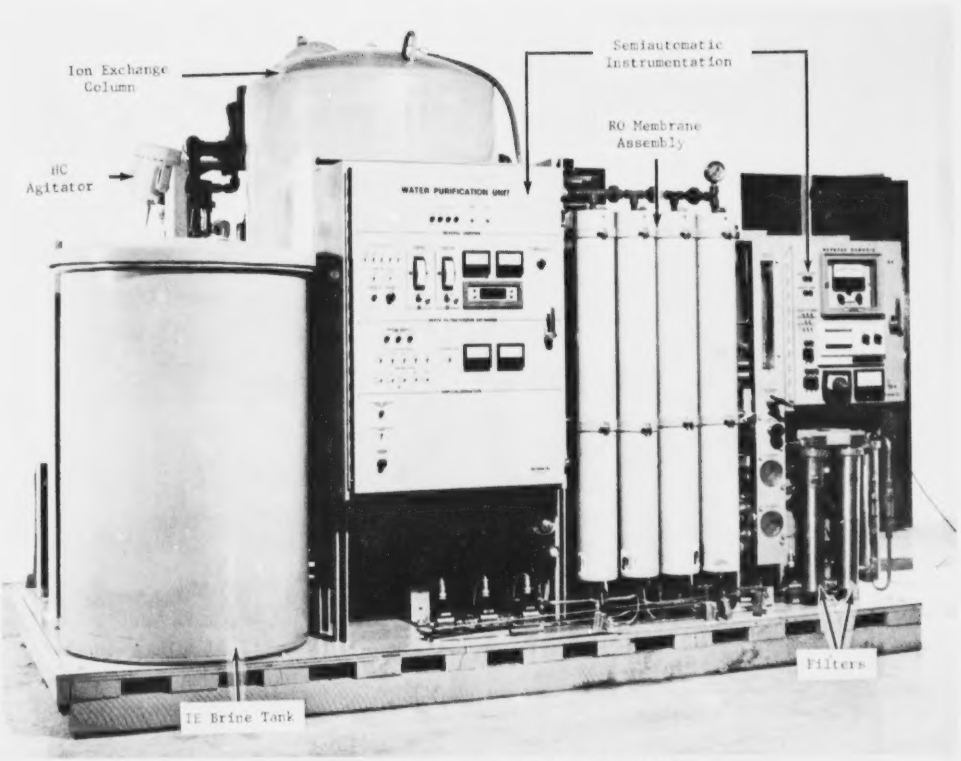


Figure 8 Water Purification Unit, Front View

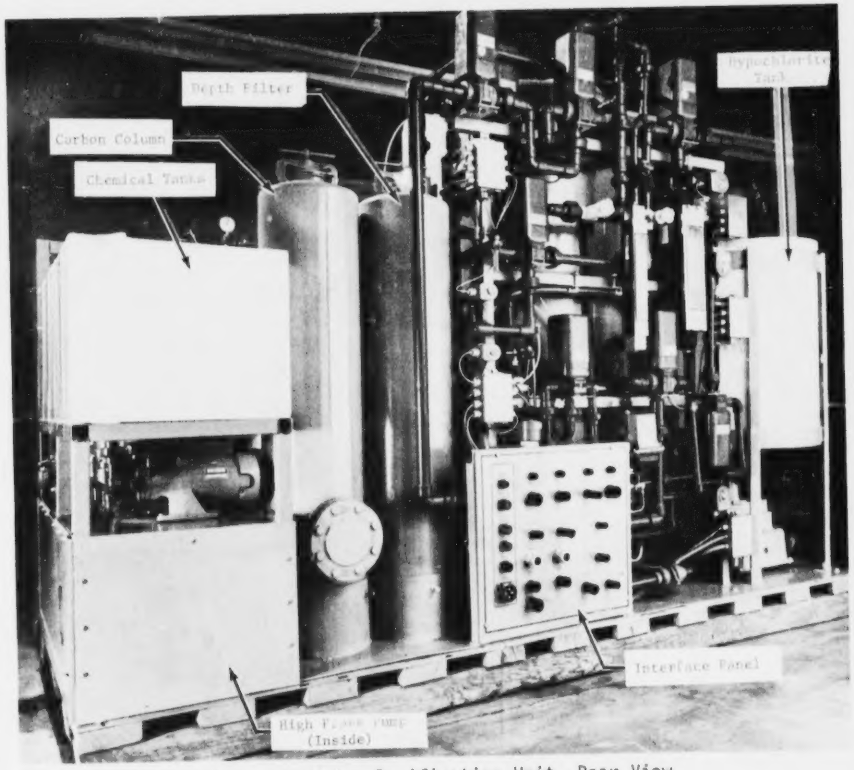


Figure 9 Water Purification Unit, Rear View

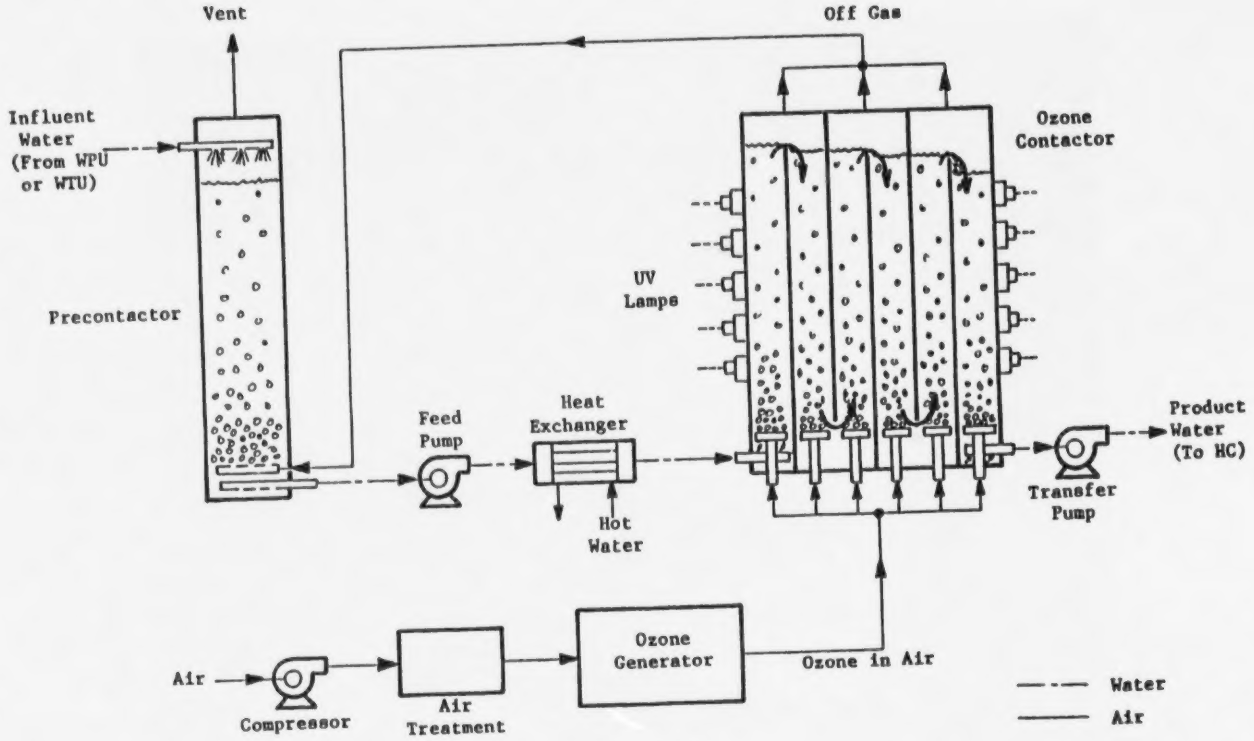


Figure 10 Flow Schematic of UV/Ozone Oxidation Unit

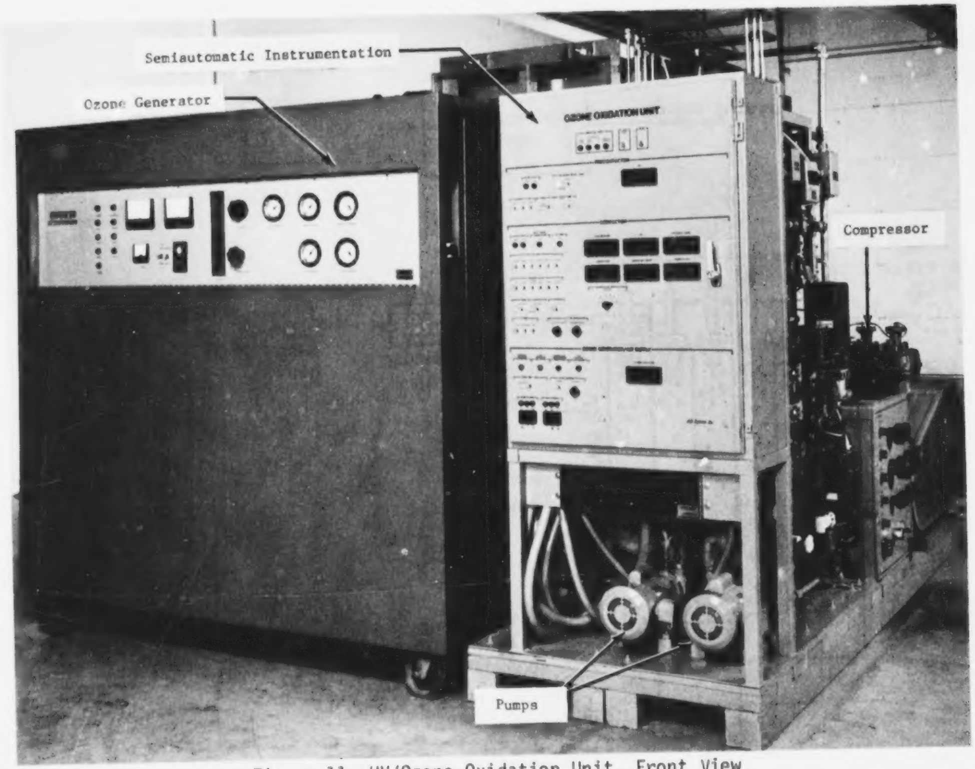


Figure 11 UV/Ozone Oxidation Unit, Front View

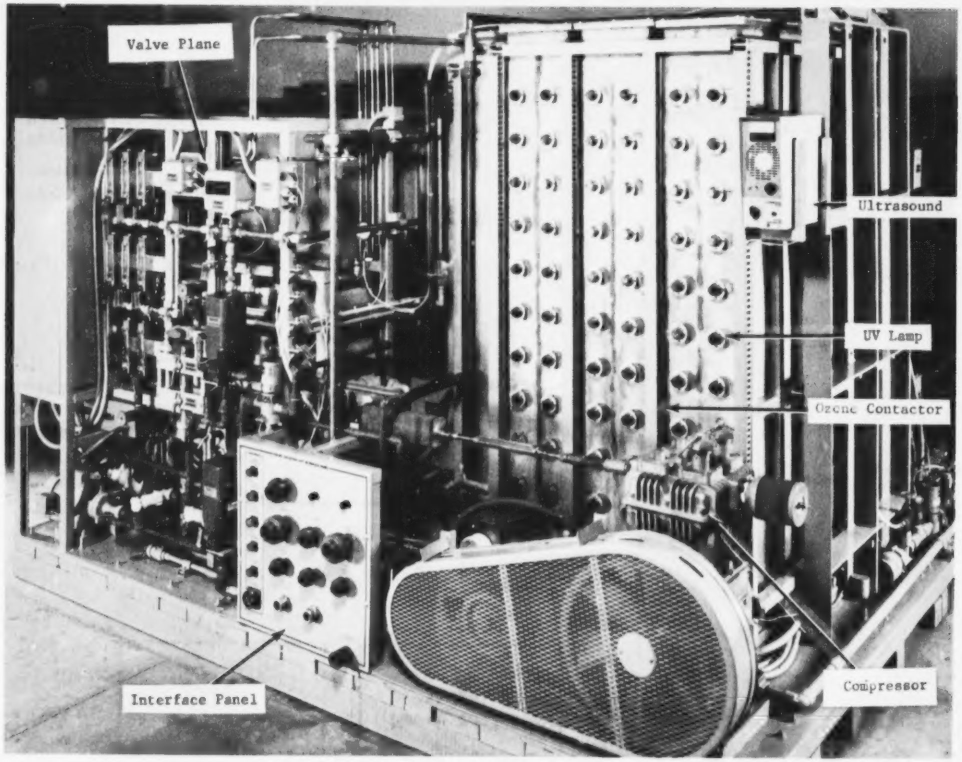


Figure 12 UV/Ozone Oxidation Unit, Side View

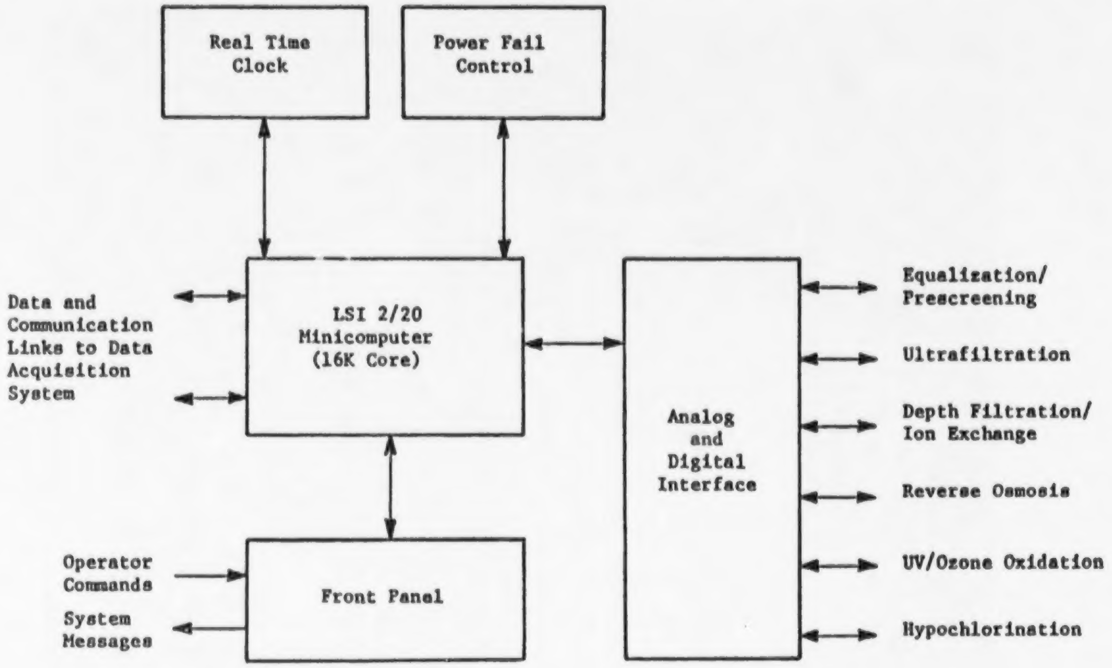


Figure 13 Automatic Instrumentation Hardware Block Diagram

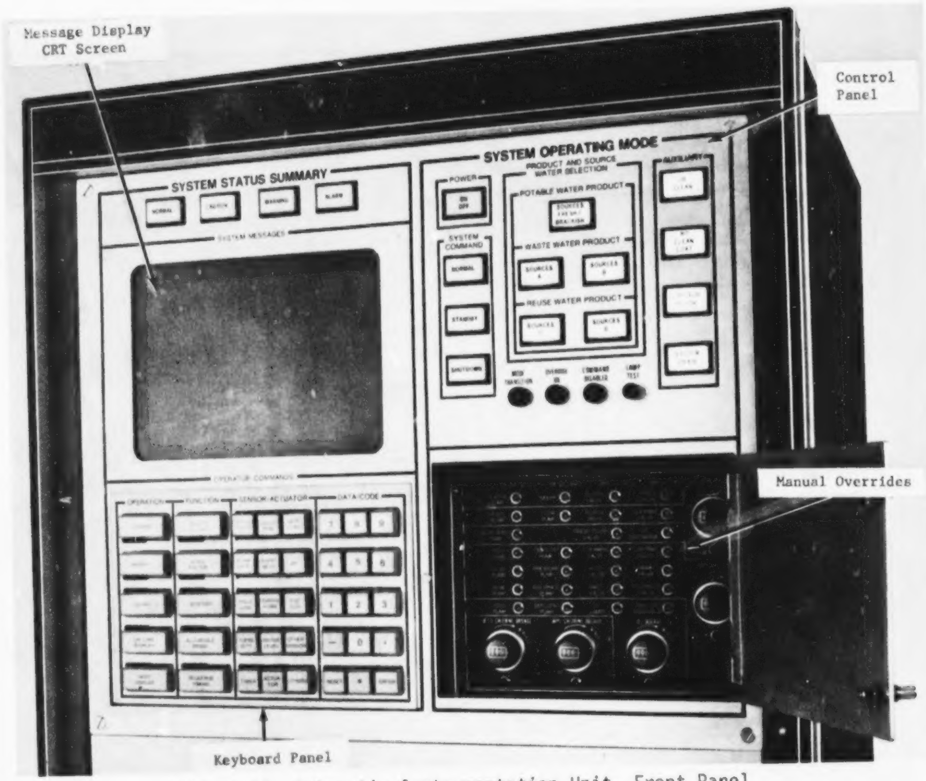


Figure 14 Automatic Instrumentation Unit, Front Panel

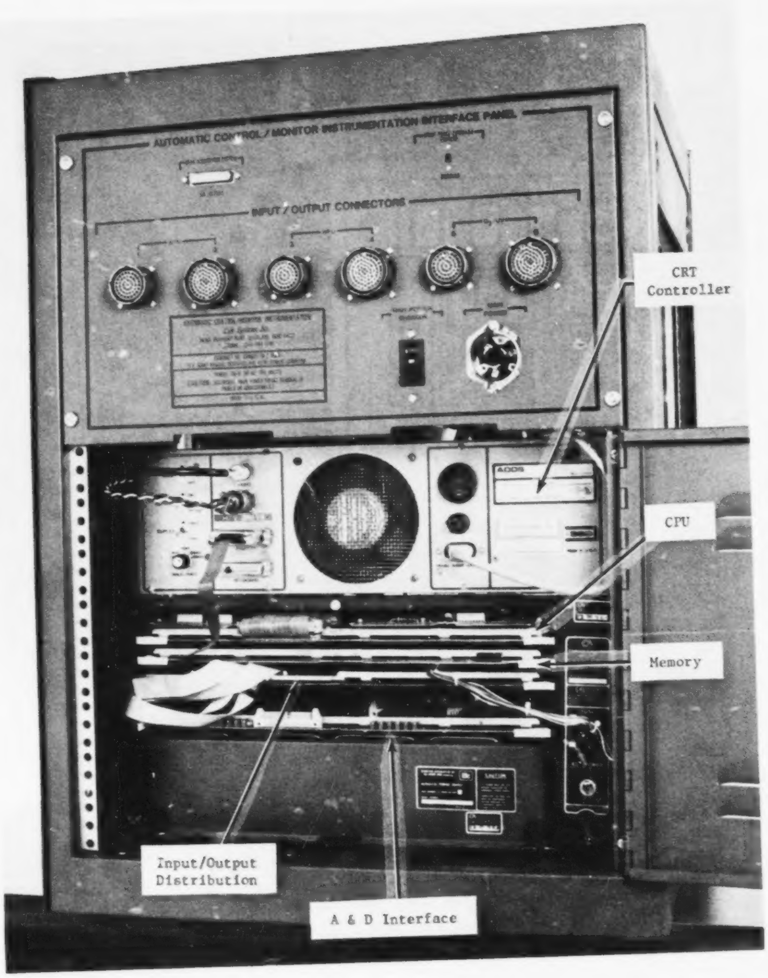

AUTOMATIC CONTROL / MONITOR INSTRUMENTATION INTERFACE PANEL
INPUT / OUTPUT CONNECTORS
CRT Controller
4005
CPU
Memory
Input/Output Distribution
A & D Interface

Figure 15 Automatic Instrumentation Unit, Interface Panel

WATER RECLAMATION FOR REUSE IN NORTHERN VIRGINIA

Millard H. Robbins, Jr.
Executive Director
Upper Occoquan Sewage Authority
P. O. Box 688
Manassas Park, Virginia 22110

George A. Gunn
Regional Manager
CH2M Hill of Virginia, Inc.
1930 Isaac Newton Sq. E.
Reston, Virginia 22090

The Occoquan Watershed is located southwest of Washington, D.C., a short distance beyond the highly developed urban area surrounding the City (Figure 1). The Watershed encompasses 600 square miles, 570 of which drain into the Occoquan Reservoir. This 1700 acre reservoir is the principal raw water source for the Fairfax County Water Authority, which serves over 660,000 thousand people in the City of Alexandria, Fairfax County and portions of Prince William County. The reservoir has a capacity of 9.8 billion gallons of water with a safe yield of 65 million gallons per day.

Despite the rapid population increase in the Washington Metropolitan area, the Occoquan Watershed until the 1960's remained largely rural. However, with the construction of the interstate highway system, the Watershed came within easy commuting distance of Washington. This afforded the persons employed in the Nation's Capital an opportunity to live in a more rural and less congested area than available in the Washington Metropolitan area, and the housing was available at much lower cost than similar housing in Washington. Consequently, pressure to expand and develop new housing in the Watershed increased tremendously.

Because of the water quality implications of a large population increase, the Virginia State Water Control Board attempted to control development by limiting the number and size of sewage treatment plants in the watershed. This policy not only had severe adverse impact on the commercial well-being of the area, but also resulted in many small, inefficient publicly and privately owned secondary sewage treatment plants with highly variable performance. These increased discharges of inadequately treated sewage resulted in the progressive deterioration of the Reservoir. By 1969, the Reservoir had heavy algal blooms, mainly blue-green algae, and the water below 10 feet in depth was anaerobic.

Recognizing that a solution to the problem required a coordinated, basin-wide comprehensive plan, the State Water Control Board employed the consulting firm of Metcalf & Eddy to develop such a plan. Following extensive studies, the consultants offered three alternate solutions to the problem.

1. Export of all wastewater out of the Occoquan Watershed.

2. Provide the most advanced methods of wastewater treatment, pipe the effluent to the Fairfax County Water Authority for direct reuse, and limit the population to produce the saleable amount of water.

3. Provide the most advanced methods of wastewater treatment and limit the sewered population to 100,000 people or a total treated wastewater capacity of 10 million gallons per day (MGD).

OCCOQUAN POLICY

Following review of the reports by the affected localities and state agencies, the State Water Control Board held a series of hearings to receive testimony from the general public. This planning effort culminated in July, 1971, with the adoption by the State Water Control Board of "A Policy for Waste Treatment and Water Quality Management in the Occoquan Watershed", commonly referred to as the Occoquan Policy.(1)

The State Water Control Board in the Occoquan Policy chose alternative 3, e.g., the most advanced waste treatment and limiting of population as the most cost effective method to protect the watershed.

Principal reasons for rejection of the other 2 alternatives were:

1. Alternative 1, the export of all wastewater, was too expensive and resulted in the transfer of a potential pollution problem to another watershed. The initial capital cost of the export plan was estimated to be $260 million with a total cost by 1990 of $442 million.

2. Alternative 2, the direct piping of the effluent to the water authority, was unacceptable to the State Health Department.

The Occoquan Policy mandated the following actions:

1. A regional water reclamation plant utilizing the best treatment technology available would replace the existing sewage treatment plants in the watershed. The effluent requirements for the regional plant were among the most stringent in the United States (Table 1).

2. The regional system would incorporate extensive fail-safe features including the following facilities at the plant and all pump stations:

(a) standby units for all electrical and mechanical components;

(b) three separate sources of electric power; and

(c) emergency holding basins.

3. Until the regional plant was available, all existing sewage treatment plants would use chemical additions to increase treatment efficiencies.

4. A watershed monitoring laboratory was required to monitor conditions of the reservoir and its tributaries.

THE UPPER OCCOQUAN SEWAGE AUTHORITY

The Upper Occoquan Sewage Authority was established in 1971 to meet the Occoquan Policy mandate to construct and operate a regional water reclamation system. The Authority selected CH2M Hill, Inc. as Consulting Engineers for the project.

The construction of the project was substantially impaired by regulatory actions subsequent to the passage of the Water Pollution Control Act amendments of 1972 (PL 92-500). These regulatory actions resulted in a very large increase in project costs and a delay in project completion of three years.(2) The projected total cost of the project is $82 million representing $21 million for sewers and pump stations and $61 million for the treatment plant However, because of the many construction inefficiencies caused by the regulatory actions, the UOSA project costs are not representative of the cost to construct such facilities in a logical manner.

A schematic of the UOSA treatment plant is presented in Figure 2. The plant is designed for an average flow of 15 MGD. The first treatment processes are fairly conventional primary and secondary treatment, with the exception of the fail-safe or redundant units provided in accordance with the Occoquan Policy. For example, each primary clarifier is rated at 7.5 MGD. Therefore, in accordance with the Policy, three primary clarifiers are provided, two clarifiers to handle the design flow of 15 MGD and a third as a redundant unit. The same redundancy is provided for all mainstream unit processes in the plant (Table 2). Another fail-safe feature in this plant section is the capability to divert a portion or all of the plant influent or primary effluent to an emergency retention pond with a capacity of 45 million gallons. The contents of this pond are returned to the plant headworks after the "emergency".

A computer based instrumentation and control system is used to monitor all critical functions in the UOSA system, and the secondary portion of the plant involves the first use of the computers for process control. The dissolved oxygen in the completely mixed activated sludge process is controlled by varying the depth of submergence of the aerator impellers. The impeller submergency is varied by computer adjustment of the aeration basin discharge weirs based on the dissolved oxygen levels.

The advanced waste treatment unit processes in the plant are normally operated in an automatic mode by the computer based central control system. However, to protect against computer failure, each system is provided with a back up analog automatic control unit. The first AWT process is chemical clarification using lime to precipitate phosphorus. Lime additions are computer controlled to raise the pH to 11.3. The precipitate is then flocculated and settled. An anionic polymer is added to enhance settling. The pH is reduced to 7.0 by two stage recarbonation with intermediate settling.

Ballast ponds, or equalization basins, are used to provide constant flows to the remaining processes. The ballast pond water is pumped through tri-media pressure filters, activated carbon adsorption columns, and ion exchange columns for ammonia removal. The water is then chlorinated and discharged to a final effluent reservoir with 20 days detention. An automatic analyzer continuously monitors the final effluent for the NPDES discharge parameters. If a parameter exceeds its limit, the computer closes the discharge valve, and the unacceptable water is recycled for additional treatment. The final effluent is the water source for all plant processes.

Organic solids are processed by two stage anaerobic digestion, dewatering on filter presses, and composting. The compost will be used by local park authorities as a soil amendment in their nurseries. The methane from digestion is used as fuel in the boilers to produce heat for the plant, and the stack gasses are used in the recarbonation process.

Chemical solids are gravity thickened and dewatered on filter presses. These solids are presently landfilled, but UOSA in cooperation with the Virginia Department of Agriculture plans to explore agricultural uses for the chemical solids.

The ion exchange beds are regenerated by purging the ammonia with a concentrated sodium chloride regenerate solution, and the ammonia is removed from the regenerant solution by an air stripping and absorption system developed for the UOSA plant. In this system, the regenerant solution pH is raised to volatilize the ammonia, ammonia gas is stripped into an air stream in a stripping tower, and the ammonia laden air stream then flows through an absorption tower. The absorbent liquid is sulfuric acid resulting in the ammonia ions being absorbed as ammonium sulfate. The ammonium sulfate will be marketed as a fertilizer.

PLANT PERFORMANCE

The UOSA plant began operations on June 26, 1978. Unfortunately, all plant construction is still not completed, and as a result some facilities are not available for use by the Authority. However, all unit processes are operative except the ion exchange facilities for ammonia removal.

We have found during our first 8 months of operation that most of the fail-safe features incorporated in the UOSA system are essential to meet the NPDES permit limits consistently. The fail-safe features were particularly valuable during the past three months. The sewage collection systems operated by the jurisdictions served by UOSA were known to have infiltration/inflow problems, but the enormity of the problem was not fully recognized until UOSA was able to intercept and treat these flows. The influence of infiltration/inflow on UOSA flows is illustrated in Figure 3. You will observe that the fairly steady flows during the Fall became highly variable during the wet months of December, January and February. During the last week in February, the flows to the UOSA plant increased from 7.8 MGD to 40.8 MG in a four day period. The actual peak flow rate was much higher, but by utilizing the emergency storage facilities at the pumping stations we reduced the peak flow to the 40.8 MG on February 25.

Despite these highly fluctuating flows, plant performance has been very good. The plant effluent characteristics are shown in Figure 4.

OPERATION AND MAINTENANCE COST

The operations and maintenance cost to date has averaged 70¢ per 1,000 gallons. When the ion exchange facilities for ammonia removal become operational, we expect the O & M cost to be about 90¢ per 1,000 gallons.

In Table 3, the UOSA operation and maintenance cost is compared to the costs incurred by some of the jurisdictions to operate their small secondary treatment plants replaced by UOSA. This comparison shows that due to economy of scale and certain efficiencies incorporated in the UOSA design, the UOSA plant is able to produce a very high quality water for a cost comparable to the cost to the jurisdictions to operate their own small secondary treatment plants.

FUTURE

The UOSA plant has already had a substantial impact on the Reservoir according to data collected by the Occoquan Watershed Monitoring Laboratory. This Laboratory, also required by the Occoquan Policy, is operated by Virginia Polytechnic Institute and State University. The Laboratory reports that the water quality in the Bull Run arm of the Reservoir last fall was the best ever observed in the six years of the monitoring program.

The quality of the UOSA water will be especially important during low flow periods of the year. In recent dry years, the water authority withdrawals exceed inflow to the Reservoir from April to October, and thus all inflow is retained in the water pool. This is illustrated in Figure 5. During an extended dry period, the wastewater discharges constitute the majority of the flow into the Reservoir and as the pool level drops, the percentage of treated wastewater in the raw water supply increases. During the 1977 drought, for example, 80 percent of the flow to the Reservoir during September was from wastewater discharges.

The impact of UOSA on the water resources of Northern Virginia is expected to increase in the future. Much of the growth in Northern Virginia during the next few years is planned for the Occoquan Watershed provided such growth can be accomplished without degrading the reservoir. Thus, UOSA is faced with the challenge of not only protecting the reservoir from wastewater pollution but serving as the source of an ever increasing supply of reclaimed wastewater for potable reuse.

References

1. Policy for Waste Treatment and Water Quality Management in the Occoquan Watershed adopted by the Virginia State Water Control Board on July 26, 1971.

2. Summary Report Concerning EPA's "Operable Unit" Requirement and Its Effects On The Capital Cost Of The UOSA Project a Position Paper adopted by the Upper Occoquan Sewage Authority Board of Directors on October 19, 1978.

TABLE 1

UPPER OCCOQUAN SEWAGE AUTHORITY WASTEWATER CHARACTERISTICS

Constituent	Expected Raw Sewage Characteristics	Required Effluent Quality
BOD_5 mg/L	250	1.0
COD mg/L	-	10.0
Suspended Solids mg/L	225	<1.0
Phosphorus (Total) mg/L	18	0.1
Nitrogen (Total Unoxidized) mg/L	25	1.0
Methylene Blue Active Substances mg/L	-	0.1
Coliform Bacteria MPN	-	<2/100 mL

TABLE 2 - PRINCIPAL FAIL-SAFE UNITS AT THE UOSA PLANT

MAJOR PROCESS	COMPONENT UNITS	IN SERVICE AT 15 MGD	REDUNDANT UNITS	DESIGN EACH
Primary Treatment	Bar Screen			
	Mechanical	1	1	45 MGD
	Manual	0	2	45 MGD
	Comminutors	1	1	39 MGD
	Parshall Flumes	1	1	45 MGD
	Sampler (Sirco)	1	0	Model BST/VS
	Clarifiers	2	1	Diameter = 125 ft. SWD = 12 ft. Weir = 392.5 ft.
Secondary Treatment	Aeration Basins	2	1	Volume = 1.89 MG BOD loading = 49.7 lbs/1000 cu.ft./day DT = 6 hrs.
	Clarifiers	3	1	Diameter = 125 ft. SWD = 12 ft. Weir = 392.5 ft.
	Flotation Thickener	1	1	500 gpm @ 5000 mg/l
Anaerobic Digestion	First Stage	1	1	1.2 MG
	Second Stage	1	0	1.1 MG
Chemical Clarification	Lime Storage Bins	1	2	80 tons
	Lime Slakers	1	2	2000 lbs/hr.
	Lime Slurry Day Tanks	1	1	750 gal.
	Rapid Mix Basins	1	1	D.T. = 4.4 min.
	Flocculation Basins	2	1	D.T. = 19 min.
	Lime Clarifiers	2	1	Diameter = 125 ft. SWD = 12 ft. Weir = 392.5 ft.

TABLE 2 (CONTINUED)

MAJOR PROCESS	COMPONENT UNITS	IN SERVICE AT 15 MGD	REDUNDANT UNITS	DESIGN EACH
Recarbonation	First Stage Basin	1	1	D.T. = 4.5 min.
	Clarifiers	2	1	Diameter = 125 ft. SWD = 12 ft. Weir = 392.5 ft.
	Second Stage Basin	1	1	D.T. = 4.5 min.
Flow Equalization	Ballast Ponds			
	Lined	1	1	1.5 MG
	Unlined	0	2	3.0 MG
	Pumps			
	Filter Supply	1	1	4.0 MGD
		1	1	8.0 MGD
	Back Wash	1	1	12.0 MGD
Filtration	Multi Media Filters	5	1	Normal 2.93 MGD @4.1 gpm/sq. ft. Extreme 3.91 MGD @5.5 gpm/sq. ft.
Carbon Adsorption	Columns with carbon regeneration furnace at 500 lbs/hr. Dry carbon loading rate	15	1	Normal contact time = 30 min. at 694 gpm (1 MGD) Extreme contact time = 22 min. at 951 gpm (1.37 MGD)
Ion Exchange	Clinoptilolite and Na^+ regeneration	6	2	Loading = 2.73 MGD @ 20 mg/l NH_4^+

TABLE 2 (CONTINUED)

MAJOR PROCESS	COMPONENT UNITS	IN SERVICE AT 15 MGD	REDUNDANT UNITS	DESIGN EACH
Chlorination	Chlorinators	1	3	2000 lbs/day
	Gravity mixers	1	1	21.8 MGD
	Contact Chambers	1	1	D.T. = 30 min.
	Analyser/recorder	1	1	

Additional special features:

* Emergency retention capacity

** at treatment plant 50 MG
** at UOSA pump stations 20 MG

* Automatic continuous analysis of effluent for pH, DO, Turbidity, MBAS, NH_3-N, Total P, TOC and conductivity.

* Electrical power sources

** two separate commercial lines
** on-site generator at plant and at each pump station

* On-site computer for monitoring and controlling process operation, computer mainframe can be run as one large unit or split into parallel operation with one side acting as in service unit and other side as a monitoring ready standby. In addition to computer control there is ability to use manual computer assisted control or an independent manually operated analog control system. The computer can present stored data in graphic or tabulated form on a CRT or a hard copy.

* In emergency conditions, plant influent can be diverted after screening to emergency retention basin. This basin is equipped with three pumps to return the retained flow to the plant headworks after the emergency has passed.

TABLE 3

COMPARISON OF OPERATION AND MAINTENANCE COSTS

Jurisdiction	Number Plants	O & M Cost per 1,000 Gallons
Fairfax County	5	$1.47
Prince William Co.	1	.43
City of Manassas	2	.71
UOSA		$ .90*

* 1978 costs

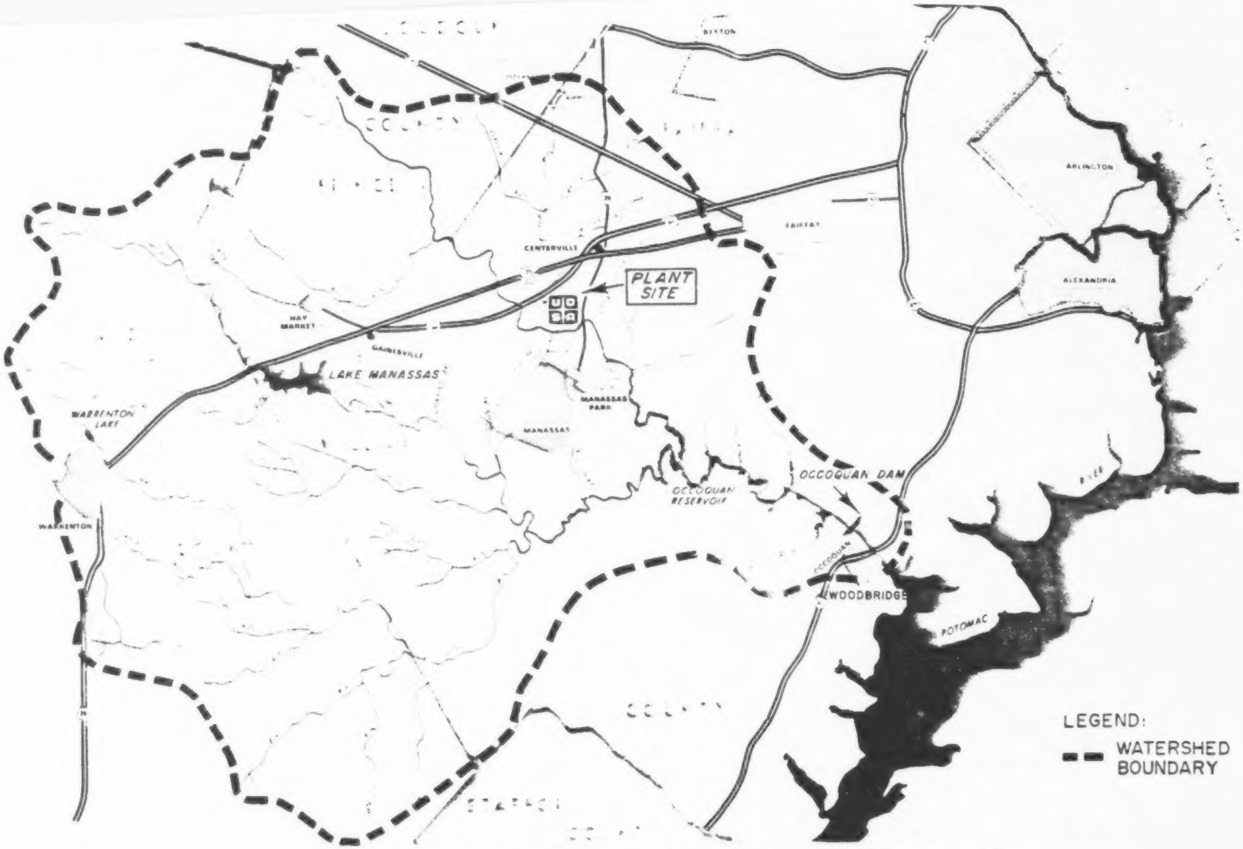
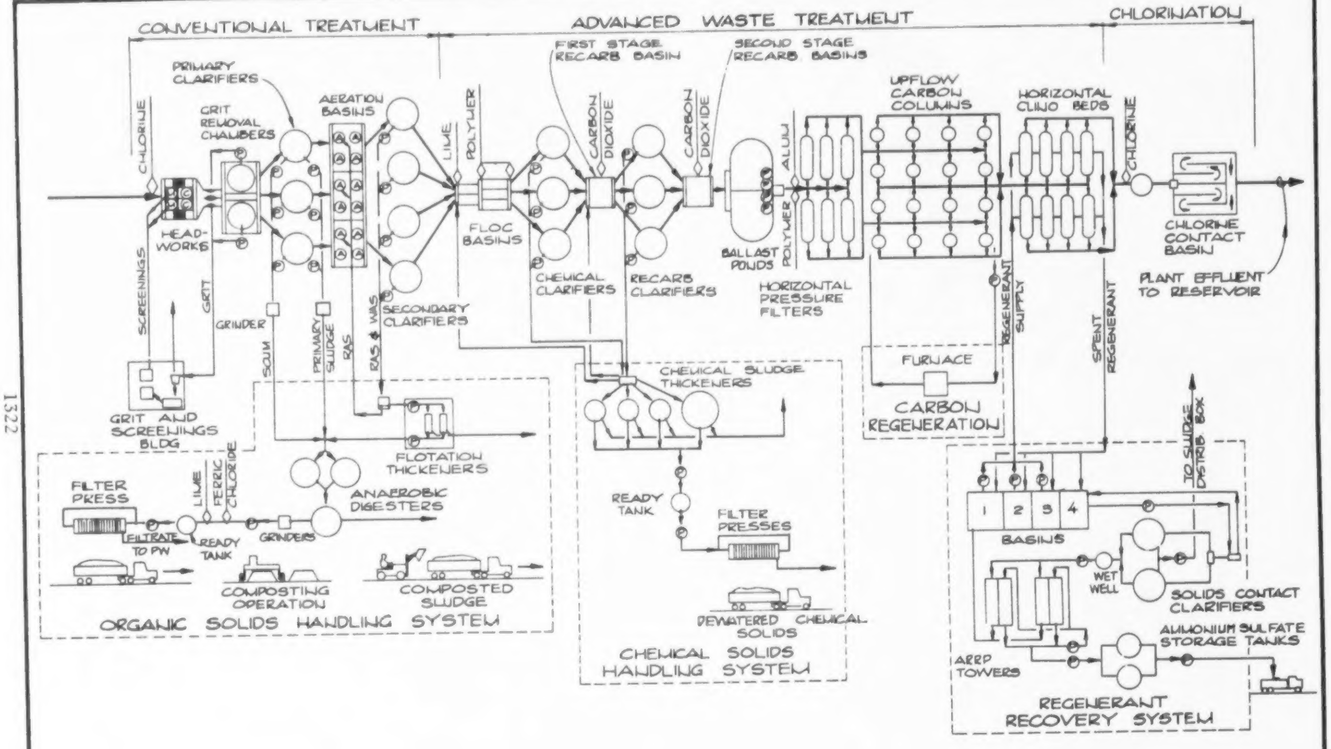


FIGURE I- GENERAL LOCATION OF OCCOQUAN WATERSHED AND UOSA PROJECT

FIGURE 2



PLANT SCHEMATIC FLOW DIAGRAM

FIGURE 3

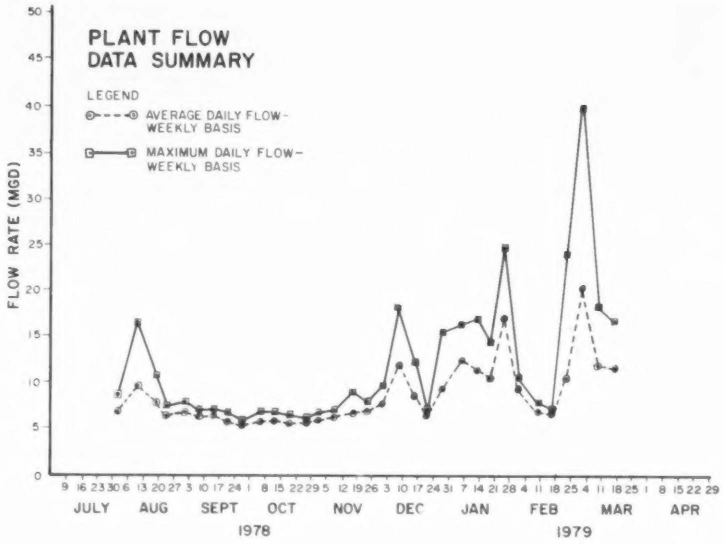

PLANT FLOW
DATA SUMMARY
LEGEND
AVERAGE DAILY FLOW-
WEEKLY BASIS
MAXIMUM DAILY FLOW-
WEEKLY BASIS
FLOW RATE (MGD)
50
45
40
35
30
25
20
15
10
5
0
9 16 23 30 6 13 20 27 3 10 17 24 1 8 15 22 29 5 12 19 26 3 10 17 24 31 7 14 21 28 4 11 18 25 4 11 18 25 1 8 15 22 29
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
1978
1979

FIGURE 4

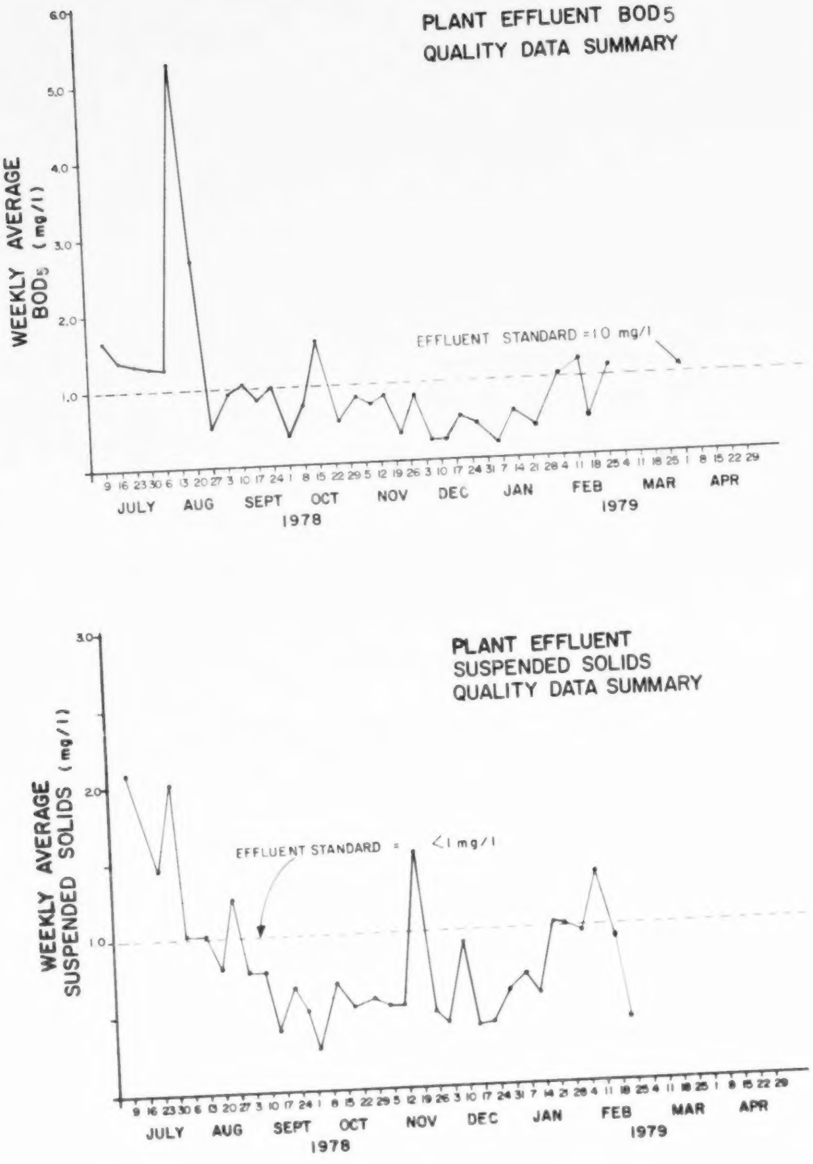
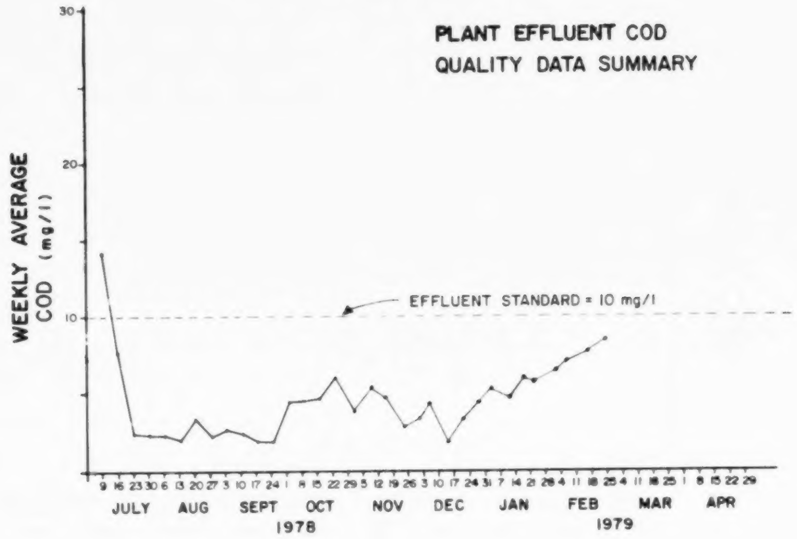

PLANT EFFLUENT BOD5
QUALITY DATA SUMMARY
WEEKLY AVERAGE
BOD5 (mg/l)
6.0
5.0
4.0
3.0
2.0
1.0
EFFLUENT STANDARD = 1.0 mg/l
9 16 23 30 6 13 20 27 3 10 17 24 1 8 15 22 29 5 12 19 26 3 10 17 24 31 7 14 21 28 4 11 18 25 4 11 18 25 1 8 15 22 29
JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR
1978
1979
PLANT EFFLUENT
SUSPENDED SOLIDS
QUALITY DATA SUMMARY
WEEKLY AVERAGE
SUSPENDED SOLIDS (mg/l)
3.0
2.0
1.0
EFFLUENT STANDARD = <1 mg/l
9 16 23 30 6 13 20 27 3 10 17 24 1 8 15 22 29 5 12 19 26 3 10 17 24 31 7 14 21 28 4 11 18 25 4 11 18 25 1 8 15 22 29
JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR
1978
1979

FIGURE 4 (CONTINUED)



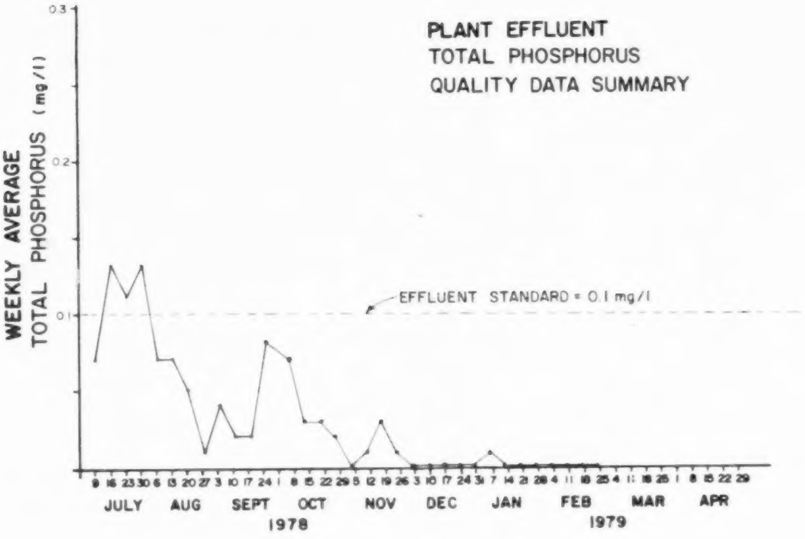
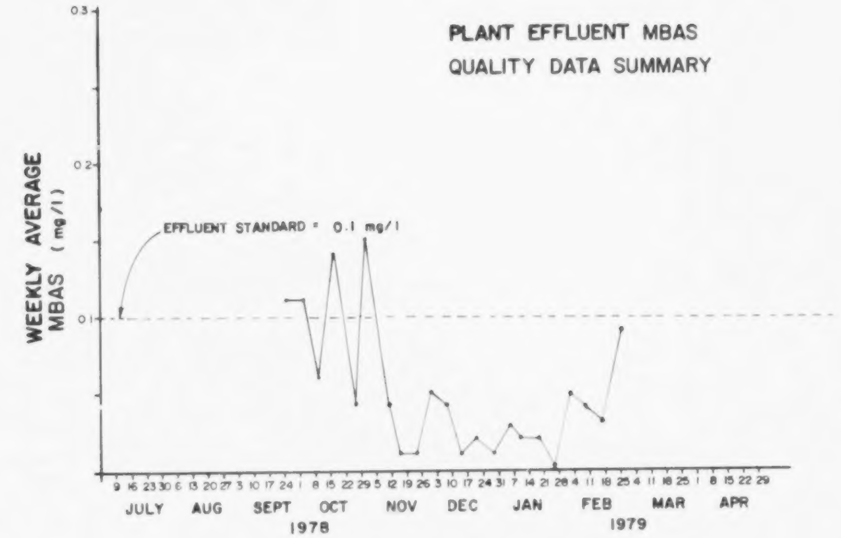
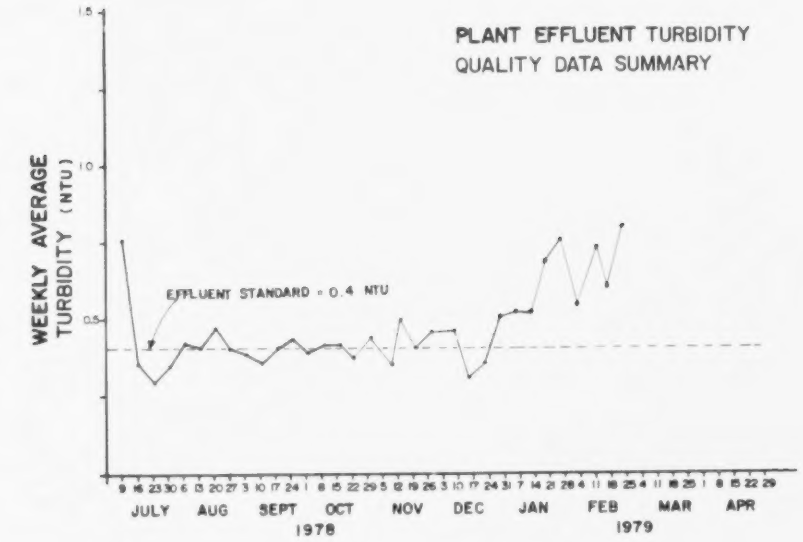


FIGURE 4 (CONTINUED)



PLANT EFFLUENT MBAS
QUALITY DATA SUMMARY

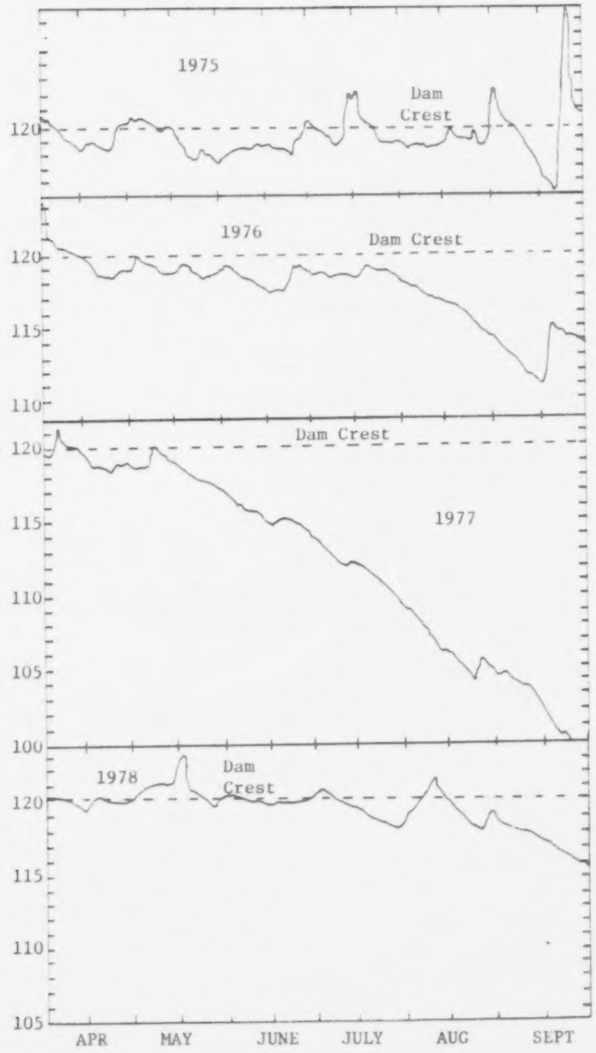


PLANT EFFLUENT TURBIDITY
QUALITY DATA SUMMARY

FIGURE 5

OCCOQUAN RESERVOIR

POOL ELEVATION, FEET ABOVE M.S.L.



TREATMENT TECHNOLOGY FOR WATER REUSE

Robert B. Williams

Justine A. Faisst

Gordon L. Culp
Culp/Wesner/Culp
El Dorado Hills, California

INTRODUCTION

The widespread and continued unplanned, indirect reuse of treated wastewaters through the discharge of these waters to streams that are subsequently used for a wide variety of purposes, has been a long accepted practice in the United States. Planned, direct reuse has been practiced on a much smaller scale for a limited number of purposes, primarily agricultural and industrial. The increasing demands on the limited water resources in the U.S. make it necessary to carefully consider multiple uses of water in water resource planning, including consideration of wastewater reuse. Evidence of the Environmental Protection Agency's interest in this area is the requirement of evaluating land application of wastewaters and wastewater reuse as part of the cost-effectiveness analysis in facility planning. Many land application projects can be considered agricultural reuse systems and categorized as planned, direct reuse of wastewater.

The main thrust of a study for the Office of Water Research and Technology, therefore, was to provide a comprehensive evaluation of the ability and the cost of available wastewater treatment processes to provide reclaimed water of suitable quality to satisfy various beneficial uses of water. This paper summarizes and presents the findings of this report titled, "Treatment Technology for Water Reuse."

The full report assesses in detail existing treatment technology for planned reuse applications. This assessment includes some of the more important and critical technical aspects of wastewater reuse systems. The technical aspects include the quality requirements for different beneficial uses, the degree and reliability of removal attained by common liquid unit processes, and the economics of various reuse treatment schemes. Other important aspects of reuse that are covered briefly include localized water demands and requirements, the natural resources available to meet these demands, and the potential for satisfying these demands. However, specific environmental issues, potential health risks, and other non-technical aspects are not addressed.

The report is concluded with a summary of the current research related to wastewater reuse. Much of the technical research is related to the application of existing treatment processes in reuse systems. This is being conducted primarily in the U.S., South Africa, and Israel. Of course, there is a great deal of concern regarding the health aspects of wastewater reuse. This has prompted numerous public health and public acceptability surveys. Two areas deserving additional research attention are the development of quality criteria for both potable and non-potable reuses and improved laboratory techniques for quick, accurate monitoring of pathogens, organics, and toxicants.

The report is intended to be used as a tool for facility planning purposes as well as to provide the necessary information to enable plant managers and engineers to assess the economic potential for reuse. The process evaluations and other information are, of necessity, generalized and should be used with care. Wherever possible, specific local information should be used for improved evaluation.

EVALUATION METHODOLOGY

The most effective method to convey the more specific information in the report is to use a particular reuse and trace the steps of the analysis. The nineteen beneficial uses identified as potential candidates for controlled reuse applications are listed in Table 1. These are presented in order of typical procurement rights although many lower level uses may actually require more stringent quality for some constituents than higher level uses.

Many publications report on specific applications of wastewater reclamation to satisfy these beneficial uses. This study considered these specific projects, but went further to present more generalized information. Using the information in the report, treatment trains can be developed to satisfy the quality requirements of virtually any of the beneficial reuses.

The contaminant removal capabilities of the treatment processes which can be used for reclamation listed in Table 2 were evaluated in detail. The data used to evaluate these processes were obtained from various full-scale and pilot test facilities across the country and are supplemented with information and data reported in the literature. The removal of various contaminants by each unit process was calculated using influent and effluent data to each unit process. This is presented in terms of percents so that it can be applied to waste with any reasonable influent contaminant concentration.

The statistical analysis was used to establish the reliability of the particular process in question. The values are given in terms of percentiles (10, 50, and 90%) which represent the percent of times a given value was achieved. For example, if the 10 percentile value for suspended solids removal in primary treatment is 75 percent, then 10 percent of the time the removal would be greater than or equal to 75 percent removal of suspended solids.

Evaluating treatment efficiencies in terms of effluent reliabilities is important in light of the current EPA definition for secondary treatment which recognizes the natural variability of treatment capability. Secondary treatment requires a 30-day average BOD and suspended solids of 30 mg/l, while the 7-day average can be as high as 45 mg/l. A recent study shows that the mean values for BOD and suspended solids should be 18 mg/l and 17 mg/l, respectively, in order to attain the 30-day values of 30 mg/l. The data presented in the report can be used to determine similar values for other contaminants and develop treatment systems with greater degrees of reliability.

In general, the performance data from wastewater treatment processes do not follow normal statistical distribution. There are many external factors and process control practices which can upset what might otherwise be a natural distribution of data. To compensate for these variations, the data can be analyzed using log-normal distribution. An example of this method is given on Figure 1. It shows the relationship between the graph drawn using the 10, 50, and 90 percentile points for the removal of suspended solids by primary treatment and the actual sample data used to derive the percentile values. The discontinuity at the 50 percentile value is characteristic of the treatment process data used in the study. The break ususally occurs between the 40 and 70 percentile values, with a marked "falling off" or decrease in process performance beyond the 90 percentile value.

Since the basic performance data in the report are for individual unit processes, a method has been presented for combining these data for the various process trains to obtain effluent quality for various levels of treatment. This is easily illustrated for the removal of suspended solids by primary and secondary treatment. The average percent removals for each process, say 50 percent for primary treatment and 80 percent for secondary treatment, are converted to decimal equivalents (0.5 and 0.8, respectively). They are then added together; the product of the two removals is then subtracted from this sum to give the overall removal in terms of a decimal equivalent, for the two processes [i.e. (0.5 + 0.8) - (0.5 x 0.8) = 0.9]. This procedure can be carried out for any number of unit processes to determine either removal or reliability. However, it must be reiterated that actual operating data should be used if they are available.

Water quality criteria for the beneficial uses were established through extensive review of available publications. The criteria finally selected were intended to be typical of the use, although the requirements could change for different areas. These criteria form the basis upon which treatment systems were developed to attain the values with a 90 percent reliability.

The level of treatment to meet the possible beneficial uses was derived from commonly used unit wastewater treatment processes as already mentioned. Because secondary treatment is now required for effluent discharge, it was considered to be the minimum degree of treatment or Level 1. The other levels were established by adding processes to meet increasingly more stringent qualities. In some cases, local requirements may prove a higher level to be the minimum acceptable treatment. These systems, summarized in Table 3, were based on successfully operating treatment facilities currently in use.

Capital and operation and maintenance costs were developed for three treatment plant sizes (1, 10, and 50 mgd) and are presented in detail in the appendix of the report. All costing was performed using a high speed computer and software package developed by CWC. The costs were presented in terms of the incremental cost to provide treatment beyond the required Level 1 quality.

Once the reuse costs were established for each beneficial use, they were compared to the costs associated with developing and transporting raw water to the same point of use. Using the values for raw water, a cost-effective treatment plant capacity can be determined. Data on the value of water or the cost of development are limited, so in several cases, general costs obtained from published papers were used. Although sludge processing is an important aspect of wastewater treatment, the specific processes selected for solids handling have little bearing on effluent quality. A basic sludge process train was selected for each wastewater treatment train for the purposes of cost and energy analysis. In some cases this was determined from a comparison of two basic alternatives (biological stabilization or physical/chemical processing), while in some cases only one type of processing was deamed appropriate and no comparison was made. Certain of the levels with extensive treatment, both biological and physical/chemical, required two separate sludge processing systems. Table 4 lists the sludge processing schemes for each level of wastewater treatment.

POTABLE WATER AND REUSE POTENTIAL

Of the possible beneficial uses listed in Table 1, the first, direct potable, is perhaps the most interesting and certainly the most controversial. It has been selected as an illustration of the methodology used in the report since it requires a high level of treatment.

Potable reuse is being practiced in several locations throughout the world, notably South Africa and Israel. Several potable reuse projects are planned in the United States. The Upper Occoquan project in Virginia will add highly treated wastewater effluent to the public raw water supply reservoir, and the Denver Water Department is undertaking a 1 mgd demonstration project. As the demand for potable water increases, the practice of potable reuse will eventually become a necessity, particularly in heavily populated and arid regions where supplies are limited. It is important to refine the technical aspects of potable reuse well in advance of their critical need.

The freshwater withdrawals in the U.S. for 1975 are shown on Figure 2. Of these, about three-quarters are from surface supplies and the remainder derived from groundwater sources. The demand for potable domestic and commercial water represented 7.5 percent of the total or about 27 billion gallons per day. As can be seen from Figure 3, the greatest minimal withdrawals were in California and the Mid-Atlantic states.

On a per capita basis, domestic use is estimated to be about 75 gallons per day (gpcpd); this includes bathing, lawn watering, and sanitary waste disposal as well as actual ingestion. Commercial, industrial, and public uses of water range from about 30 to 130 gpcpd, depending on local conditions. For general planning purposes, 160 gpcpd is used as a typical figure for overall municipal potable water consumption.

The quality criteria for the potential beneficial uses has been derived from various standards and accepted practice. Quality criteria were examined for all of the previously listed beneficial uses. In the case of potable use, the requirements were established with the passage of the Safe Drinking Water Act in 1974. Interim primary drinking water regulations (IPDWR) listed in Table 5 were established by the EPA and became effective on June 24, 1977. They are subject to revision before becoming final and may be superceded by more stringent state or local standards. The secondary standards pertain to aesthetics rather than health. The adoption of the trihalomethane limitation of 0.10 mg/l is still in question since meeting this MCL requires processing not ordinarily used in water treated (granular activated carbon) and carries with it significant costs.

The IPDWR represent standards to be met regardless of the source of supply. As the prospect of potable reuse comes closer to being a reality, there is some concern that raw water criteria should be regulated. There is particular concern with groundwater supplies subject to recharge by direct injection or land spreading. Historically, the quality of raw groundwater has been much higher than that of surface water; therefore, treatment prior to use has been minimal (disinfection only in most cases). The extent and reliability of treatment by percolation through the soil mantle over an aquifer may necessitate strict raw water quality criteria in addition to finished product requirements.

RENOVATING WASTEWATER FOR POTABLE REUSE

The treatment level necessary to meet the IPDWR's is illustrated by the treatment train on Figure 4. It is comprised of primary, biological secondary (activated sludge), lime coagulation-flocculation-sedimentation, recarbonation, mixed media filtration, selective ion exchange, breakpoint chlorination, granular activated carbon adsorption, reverse osmosis, ozonation, and finally, residual chlorination.

The average performance for this train is given in Table 6 in terms of percent removal, reliability, and effluent concentrations for a set of basic constituents. These values were derived from statistical analyses of individual unit process performance data. Removals were calculated simply using influent and effluent data to each unit process; however, similar information is not available for pesticides. This information shows that the basic quality requirements for potable use can be met by the proposed process train. However, because potable reuse is a sensitive issue, special removal considerations have also been addressed. These concern the removal of pathogens, organics (chloroform, trihalomethanes, pesticides, etc.), and heavy metals.

As noted in the process train schematic, Figure 4, ozonation has been included for disinfection; it is followed by chlorination to provide residual disinfection. Activated carbon adsorption either before or after chlorination has been proven effective in removing both chlorinated organic compounds and persistent organic pesticides; in the process flow schematic, it follows breakpoint chlorination, which has been included for residual ammonia removal. Heavy metal removal is effectively achieved by chemical coagulation-sedimentation. Insoluble metal oxides and hydroxides are formed at a pH of about 11; this is achieved by lime addition, which also serves to remove phosphorous from the waste stream.

ECONOMIC ASSESSMENT OF WATER REUSE

The question of costs has always been of interest in the planning of a new project; recently, energy consumption has received increasing attention. To facilitate the cost-effectiveness analysis a computer program was utilized to derive costs and energy estimates for the various treatment trains. The information stored in the computer includes curves for capital and maintenance materials costs. These are keyed to Bureau of Labor Statistics (BLS) cost indices for construction materials, labor, etc., so they can be updated as prices rise. Certain costs, such as labor, electrical energy, and fuel are updated using fixed unit prices. The program automatically calculates construction, annual O & M, amortized capital, and unit treatment costs (i.e., annual cost/dwelling, cents/thousand gallons) depending on the variables entered in the input (i.e., contingencies, interest rate, amortization period, population per dwelling).

The costs for each of the unit processes in the train are then totalled to give the treatment level costs. A sample of the output for Level 11 (potable water) is given on Figure 5. The use of the computer facilitated comparison of minor variations in a particular treatment train such as chlorination versus ozonation for disinfection. Basic sludge processing trains were also costed so that the economic evaluation would be complete. Biological and physical/chemical stabilization systems were analyzed as appropriate. Where solids hauling was needed, the computer analysis made comparing haul distances simple.

The cost information in the report was also presented graphically to aid in comparing the cost of raw water supply with that of various levels of reuse. In the report, the basic costs for Level 1 treatment (secondary) were given. For higher levels of treatment, only the incremental values were given. As mentioned previously, secondary treatment is considered the minimum wastewater treatment and would be required regardless of any reuse projects. The incremental cost curves provide a quick basis for comparing the additional cost of upgraded treatment required for various reuses.

The examples presented herein are totals for Level 11 treatment. These were derived quite simply by adding the incremental costs to the basic Level 1 curves. They also include the most economical methods for sludge processing.

Figure 6 illustrates the annual costs in terms of dollars per million gallons. In cases of such high levels of treatment, the annual capital and O & M costs are roughly equal. For lower levels, it was found that the O & M costs were somewhat less than the amortized capital costs. The economies of scale are quite apparent from the curves, particularly between one and ten mgd. Figures 7 and 8 show the total annual energy requirements (both primary and secondary) and the labor and materials requirements, respectively. Labor and maintenance materials were used in calculating the annual costs using current unit prices.

The economic feasibility of reuse was assessed for each beneficial use using similar curves. A general summary of the limits of annual costs is given on Figure 9. It shows the envelope of wastewater treatment costs bound by the extreme Levels 1 and 11 (Levels 12 and 13 involve land application and physical/chemical treatment which are not directly comparable to trains incorporating biological processes.) The range of raw water costs is bound on the low end by $3/mil gal for an abundant supply of good quality agricultural irrigation water, and at the high end by the projected $1,600/mil gal for drinking water in Denver, which includes development and treatment.

To assess the economic feasibility of direct potable reuse, the cost of Level 11 treatment is compared with the cost of raw water treatment. These values are presented graphically for various types of treatment in Figure 10 along with the incremental cost for Level 11 treatment. It is apparent that the cost of raw water treatment, even at a high level with reverse osmosis, is still less than producing potable water from wastewater. However, in water-short areas fresh water in the Grand Canyon Village (where a sub-potable supply is used for sanitary purposes) is close to $2,500/mil gal; sub-potable water is sold for $1,000 to $1,750/mil gal. In other arid areas, the cost of delivered water ranges from $600 to $1,200/mil gal. As the cost of developing and delivering new supplies increases, the economic feasibility of direct potable reuse will also increase.

CONCLUSIONS

Although this paper has emphasized direct potable reuse, many other reuses were also studied. Under present conditions, it was found that numerous beneficial uses, many of which require large volumes of water, may be satisfactorily supplied by properly treated wastewater. As indicated in Table 7, uses requiring only secondary effluent include agricultural irrigation, livestock and wildlife watering, once-through cooling, and the primary metals industries. Landscape and urban irrigation waters can be supplied by filtered secondary effluent as can the water used by the petroleum industries for secondary oil recovery. Higher qualities of water used for recycle cooling and boiler make-up are economically feasible based on the purchase price of water with an allowance given for the treatment costs by industry. Beneficial uses that do not appear economically feasible include warm and cold water fisheries and primary and secondary recreation. For these uses, the quality requirements are high and the treatment is costly. From Table 8, it can be seen that costs for these uses range from 55¢ to $1.10 per 1,000 gal at the 10 mgd plant capacity.

Of course, the feasibility of any reuse project is subject to local conditions. Water-short areas will be more likely to have successful reuse projects than areas where there is an abundance (or at least a sufficiency) of good quality fresh water. The proximity of the wastewater treatment facility and the demand also has a bearing on the viability of reuse, particularly if the raw water delivery has high transport costs associated with it. It cannot be overstressed that critical analysis of local conditions will be the deciding factor on whether or not a reuse program should be undertaken.

ACKNOWLEDGMENTS

This report upon which this paper was based was prepared under the direction of Mr. Robert Madancy of the U.S. Department of the Interior, Office of Water Research and Technology, Washington, D.C. The work was conducted under Contract Number: 14-34-0001-7803.

We would also like to thank the many individuals and agencies who provided the daily operating reports and data for their respective treatment facilities. These data found the basis of the probability analyses of the treatment processes presented in the full report.

BIBLIOGRAPHY

1. Culp-Wesner-Culp, "Water Reuse and Recycling, Volume 1; Evaluation of Needs and Potential," prepared for the U.S. Department of the Interior, Office of Water Research and Technology, prepublication volume, OWRT/RU-79/1.
2. Culp-Wesner-Culp, "Water Reuse and Recycling, Volume 2; Evaluation of Treatment Technology," prepared for the U.S. Department of the Interior, Office of Water Research and Technology, prepublication volume, OWRT/RU-79/1.

TABLE 1
Potential Beneficial Uses

- Direct Potable
- Public Water Supply -
    - Surface Water
    - Groundwater
- Agricultural Irrigation -
    - Forage Crops
    - Truck Crops
- Urban or Landscape Irrigation
- Livestock and Wildlife Watering
- Power Plant and Industrial Cooling -
    - Once-through
    - Recirculation
- Industrial Boiler Make-up
- Industrial Water Supply -
    - Food and Kindred Produts
    - Paper and Allied Products
    - Chemical and Allied Products
    - Petroleum and Allied Products
    - Primary Metals
- Primary Contact Recreation
- Secondary Contact Recreation
- Warm Water Fisheries
- Cold Water Fisheries

TABLE 2
Treatment Process Used for Reclamation

Process	Principal Contaminants Removed
Primary Sedimentation	TSS, BOD, grease & oil
Activated Sludge	BOD, COD, TSS, grease & oil, some heavy metals
Nitrification	BOD, COD, TSS, ammonia, some heavy metals
Denitrification	Nitrate
Trickling Filter	BOD, COD, TSS
Rotating Biological Contactor	BOD, COD, TSS, ammonia, some heavy metals
Filtration	TSS, turbidity
Activated Carbon Adsorption	BOD, TSS, colifroms, TOC
Chemical Coagulation-Flocculation	BOD, COD, TSS, phosphorous, some heavy metals
Ammonia Stripping	Ammonia
Selective Ion Exchange	Ammonia, TSS
Reverse Osmosis	TDS
Chlorination	Coliforms
Ozonation	Coliforms,color, turbidity
Land Treatment	BOD, COD, TSS, ammonia, phosphorous, oil & grease, coliforms

TABLE 3
Treatment Levels

Treatment Level	Treatment System
1a	Activated sludge
1b	Trickling filter
1c	Rotating biological contactors
2a	2-Stage nitrification
2b	Rotating biological contactors
2c	Extended aeration
3a	Nitrification-Denitrification
3b	Selective ion exchange
4	Filtration of secondary effluent
5a	Alum added to aeration basin
5b	Ferric chloride added to primary
5c	Tertiary lime treatment
6a	Tertiary lime, nitrified effluent
6b	Tertiary lime plus ion exchange
7	Carbon adsortion, filtered secondary effluent
8	Carbon, tertiary lime effluent
9	Carbon, tertiary lime, nitrified effluent
10	Carbon, tertiary lime, ion exchange
11	Reverse osmosis of AWT effluent
12a	Physical-Chemical system, lime
12b	Physical-Chemical system, ferric chloride
13a	Irrigation
13b	Infiltration-Percolation
13c	Overland flow

TABLE 4
Sludge Processing Alternatives For Various Treatment Levels

	Treatment Level	Sludge Processing
1a.	Activated Sludge	Anaerobic Digestion or Chemical Processing
1b.	Trickling Filters	Anaerobic Digestion
1c.	Rotating Biological Contactors	Anaerobic Digestion
2a.	2-Stage Nitrification	Anaerobic Digestion
2b.	Rotating Biological Contactors	Anaerobic Digestion
2c.	Extended Aeration	Aerobic Digestion (included in wastewater treatment)
3a.	Nitrification-Denitrification	Anaerobic Digestion
3b.	Selective Ion Exchange	Anaerobic Digestion
4.	Filtration of Secondary Effluent	Anaerobic Digestion
5a.	Alum Added to Aeration Basin	Anaerobic Digestion
5b.	Ferric Chloride Added to Primary	Anaerobic Digestion or Chemical Processing
5c.	Tertiary Lime Treatment	Anaerobic Digestion of Organic Sludge and Chemical Processing of Chemical Sludge
6a.	Tertiary Lime, Nitrified Effluent	Anaerobic Digestion of Organic Sludge and Chemical Processing of Chemical Sludge
6b.	Tertiary Lime Plus Ion Exchange	Anaerobic Digestion of Organic Sludge and Chemical Processing of Chemical Sludge
7.	Carbon Adsorption, Filtered Secondary Effluent	Anaerobic Digestion
8.	Carbon, Tertiary Lime Effluent	Anaerobic Digestion and Chemical Processing of Lime Sludge
9.	Carbon Tertiary Lime, Nitrified Effluent	Anaerobic Digestion of Organic Sludge and Chemical Processing of Chemical Sludge
10.	Carbon Tertiary Lime, Ion Exchange	Anaerobic Digestion and Chemical Processing of Lime Sludge
11.	Reverse Osmosis of AWT	Anaerobic Digestion and Chemical Processing of Lime Sludge
12a.	Physical-Chemical System, Lime	Chemical Processing
12b.	Physical-Chemical System, Ferric Chloride	Chemical Processing

TABLE 5
Drinking Water Quality
(all units in mg/l unless otherwise noted)

Parameters	Drinking Water MCL
Primary Regulations	
Arsenic	0.05
Barium	1
Cadmium	0.010
Chromium	0.05
Fluoride	1.4 to 2.4*
Lead	0.05
Mercury	0.002
Nitrate (as N)	10
Selenium	0.01
Silver	0.05
Radium, p Ci/l	5
Endrin	0.002
Lindane	0.004
Methoxychlor	0.01
Toxaphene	0.005
2, 4-D	0.01
2, 4, 5-TP Silvex	0.01
Turbidity, TU	1**
Coliform Bacteria (colonies/100 ml)	1+
Secondary Regulations	
Copper	1
Iron	0.03
Manganese	0.05
Sulphate	250
Zinc	5
Color, units	15
Foaming Agents (as MBAS)	0.05
Odor, TON	3
Other	
Trihalomethane	0.10-

*Varies with average annual maximum daily air temperature

**Monthly average

+Monthly average, membrane filter technique

-Proposed MCL

TABLE 6
LEVEL II Treatment - Carbon Adsorption of Lime Treated Activated Sludge Effluent - Average Process Train Performance

Constituent	Average Removal, %	Average Reliability 10%	50%	90%	Average Effluent Concentration, mg/l
BOD	100	100	100	89	0
COD	100	100	100	97	0
TSS	100	100	99	87	0
NH_3-N	100	97	81	48	0
Phosphorous	100	100	100	99	0
Oil & Grease	97	100	98	73	2
Arsenic	61	93	63	0	.003
Barium	79	95	79	52	.092
Cadmium	98	100	98	87	.0002
Chromium	100	100	98	84	0
Copper	98	100	99	98	.002
Fluoride	x	x	x	x	x
Iron	99	100	100	94	.023
Lead	99	100	98	78	.001
Manganese	98	100	98	86	.002
Mercury	23	31	18	0	.028
Selenium	7	26	12	0	.006
Silver	82	100	99	80	.004
Zinc	98	100	95	58	.008
TOC	100	100	98	83	0
Turbidity	100	100	100	95	0*
Color	93	100	94	56	5**
Foaming Agents	92	-	84	-	.17
TDS	95	-	-	-	129

---

*TU
**P-C Units

SYMBOLS: x - data inconclusive
- - insufficient data

TABLE 7

Treatment Levels Required For Various Beneficial Uses

Beneficial Use	Treatment Levels
Agricultural Irrigation - Forage Crops	1
Agricultural Irrigation - Truck Crops	1
Urban Irrigation - Landscape	4
Livestock and Wildlife Watering	1
Power Plant and Industrial Cooling	
Once-through	1
Recirculation	5c or 6b
Industrial Boiler Make-up, low pressure	6
, intermediate pressure	10
Industrial Water Supply	
Petroleum and Coal Products	3a or 4
Primary Metals	1
Paper and Allied Products	5c or 8
Chemicals and Allied Products	7 to 11
Food and Kindred Products	7 to 11
Fisheries	
Warm Water	6
Cold Water	6
Recreation	
Secondary Contact	4
Primary Contact	5 or 6
Public Water Supply	
Groundwater, spreading	13b
, injection	11
Surface Water	9, 10, or 12
Direct Potable	11

TABLE 8
Summary of Unit Treatment Costs

Treatment Level	Treatment System	Unit Cost, ¢/1000 gal 1 mgd	10 mgd	50 mgd
1a	Activated sludge	100.0	47.4	36.3
1b	Trickling filter	104.7	52.6	39.5
1c	Rotating biological contactors	107.2	65.1	55.8
2a	2-Stage nitrification	130.7	58.8	44.7
2b	Rotating biological contactors	142.3	98.2	85.0
2c	Extended aeration	43.2	28.4	-
3a	Nitrification-Denitrification	150.2	73.2	56.0
3b	Selective ion exchange	189.3	89.9	65.5
4	Filtration of secondary effluent	132.8	55.1	42.7
5a	Alum added to aeration basin	172.8	70.3	57.4
5b	Ferric chloride added to primary	156.4	77.5	57.8
5c	Tertiary lime treatment	196.1	81.7	59.3
6a	Tertiary lime, nitrified effluent	217.0	83.0	60.6
6b	Tertiary lime plus ion exchange	224.4	108.9	77.2
7	Carbon adsorption, filtered secondary effluent	176.5	72.7	57.2
8	Carbon, tertiary lime effluent	238.7	99.4	74.2
9	Carbon, tertiary lime, nitrified effluent	263.8	100.6	75.5
10	Carbon, tertiary lime, ion exchange	288.0	126.5	91.9
11	Reverse osmosis of AWT effluent	481.3	214.6	171.8
12a	Physical-Chemical system, lime	250.4	103.3	81.2
12b	Physical-Chemical system, ferric chloride	255.6	115.0	86.4
13a	Irrigation	79.5	61.5	53.5
13b	Infiltration-Percolation	38.1	19.7	15.8
13c	Overland flow	52.4	35.4	30.1

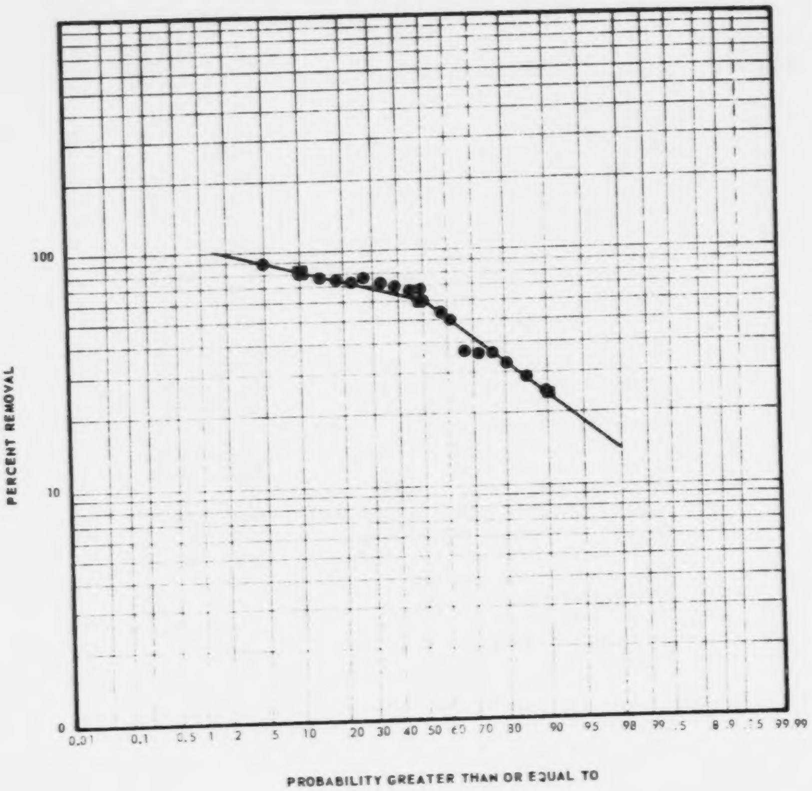


Figure 1. PROBABILITY DISTRIBUTION OF SUSPENDED SOLIDS REMOVAL IN PRIMARY TREATMENT PROCESS

(Data source: San Francisco Wastewater Treatment Pilot Plant Study)

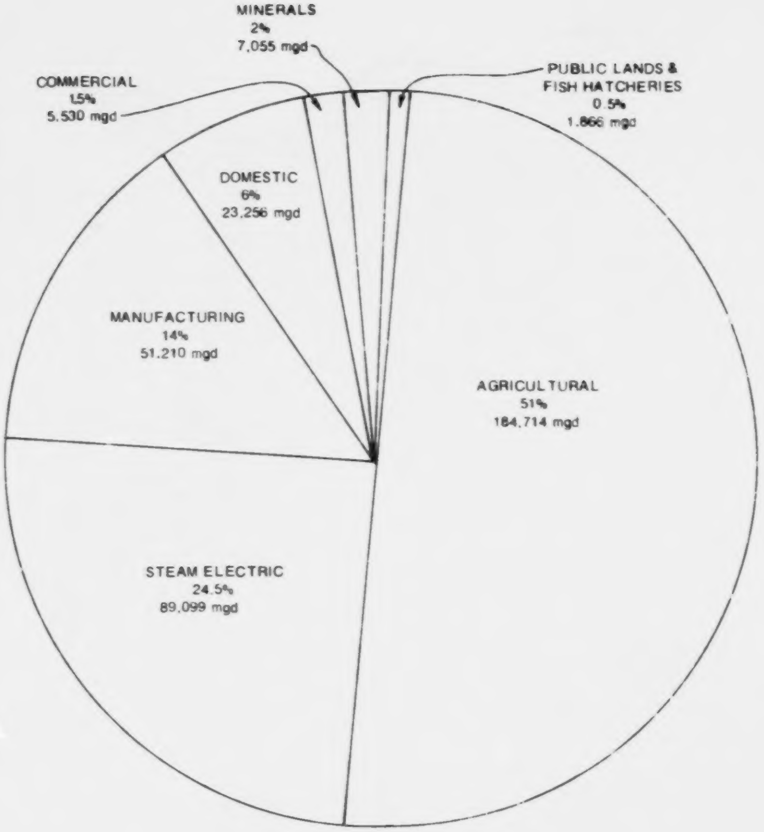


Figure 2.
NATIONAL FRESH WATER WITHDRAWALS – 1975

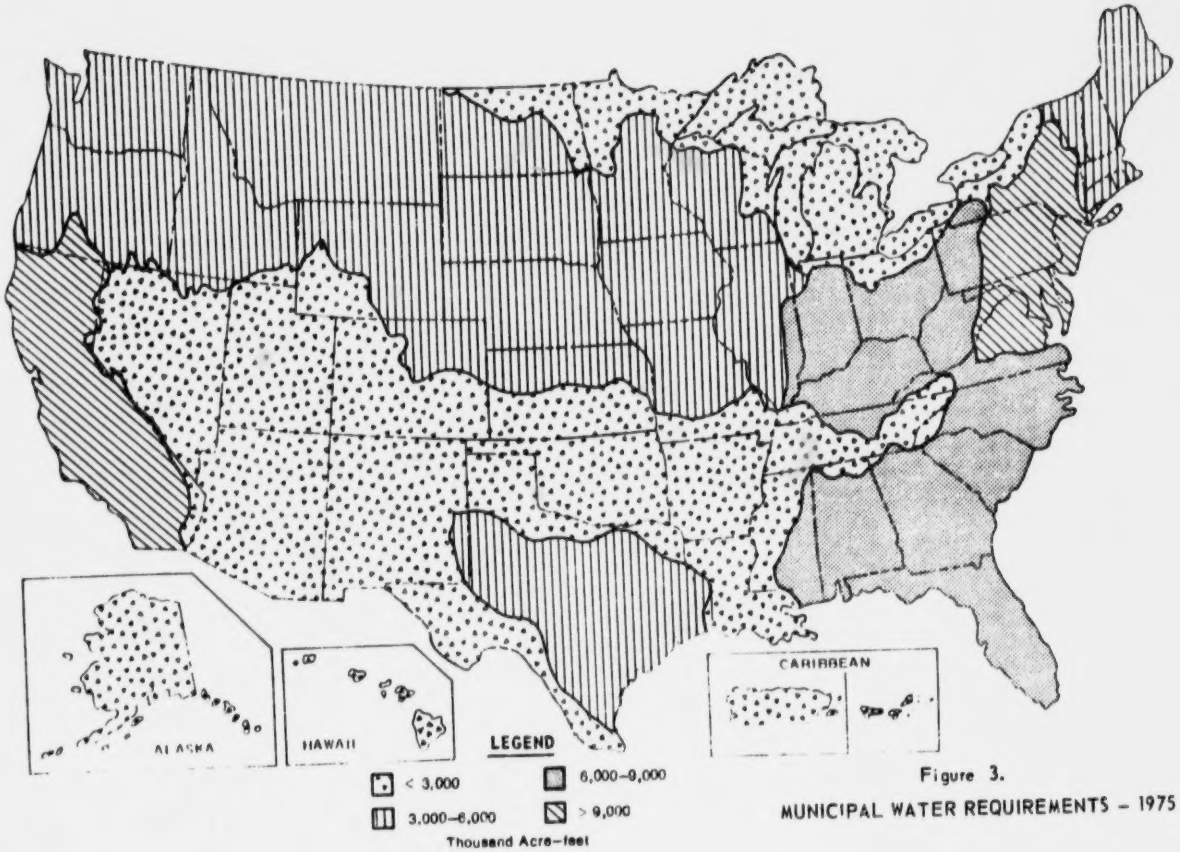


Figure 3.

MUNICIPAL WATER REQUIREMENTS – 1975

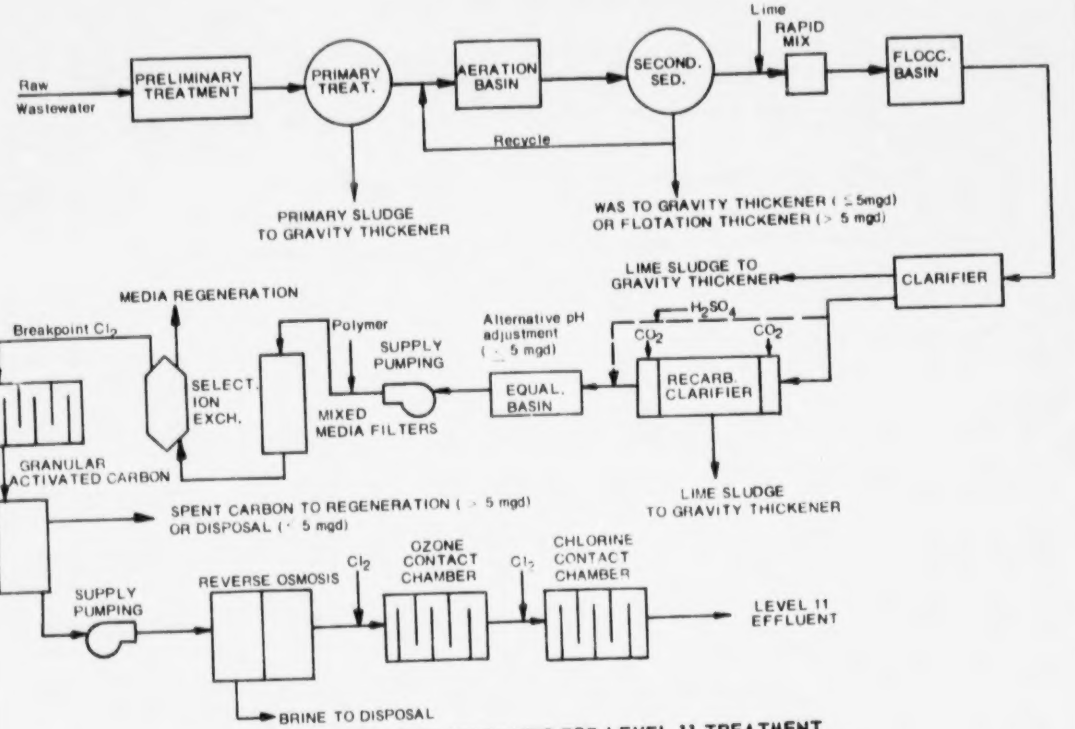
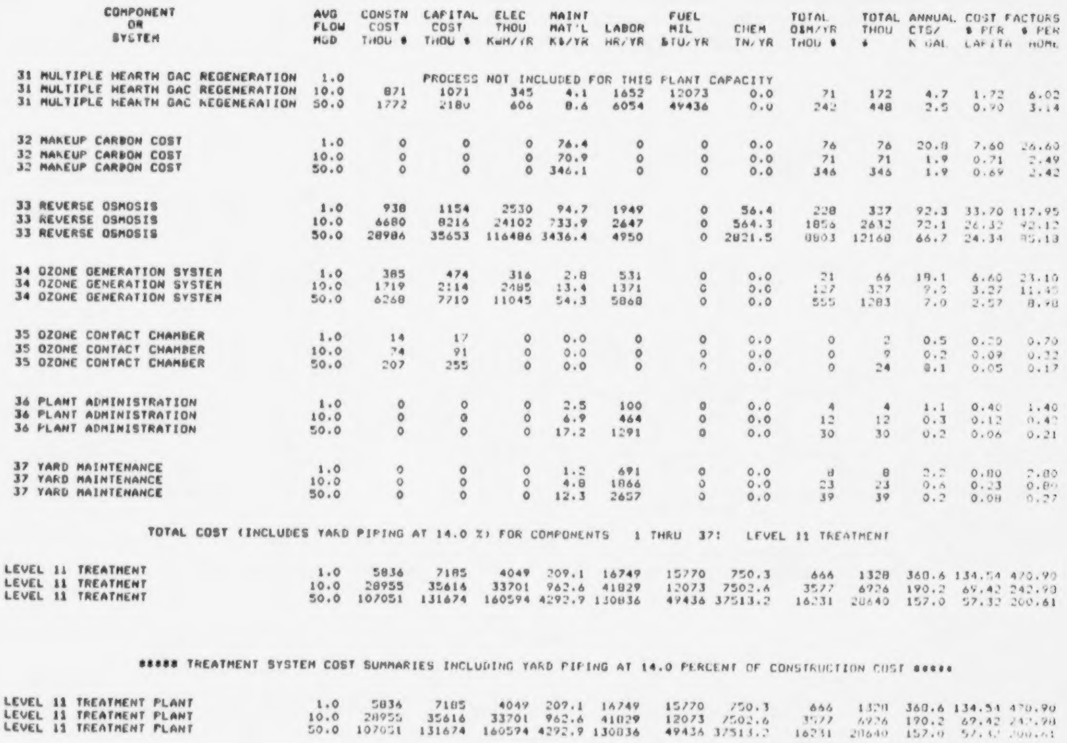


Figure 4 PROCESS SCHEMATIC FOR LEVEL 11 TREATMENT REVERSE OSMOSIS FOR VERY LOW COD AND DEMINERALIZATION



```
              COMPONENT                AVG   CONSTN  CAPITAL  ELEC    MAINT           FUEL             TOTAL    TOTAL ANNUAL COST FACTORS
                 OR                    FLOW   COST    COST    THOU    MAT'L   LABOR   MIL      CHEM    O&M/YR   THOU  CTS/   $ PER  $ PER
               SYSTEM                  MGD   THOU $  THOU $  KWH/YR  K$/YR   HR/YR   BTU/YR   TN/YR   THOU $   $     K GAL  CAPITA  HOME

31 MULTIPLE HEARTH GAC REGENERATION    1.0           PROCESS NOT INCLUDED FOR THIS PLANT CAPACITY
31 MULTIPLE HEARTH GAC REGENERATION   10.0     871    1071     345     4.1    1652   12073     0.0      71     172     4.7   1.72   6.02
31 MULTIPLE HEARTH GAC REGENERATION   50.0    1772    2180     606     8.6    6054   49436     0.0     242     448     2.5   0.90   3.14

32 MAKEUP CARBON COST                  1.0       0       0       0    76.4       0       0     0.0      76      76    20.8   7.60  26.60
32 MAKEUP CARBON COST                 10.0       0       0       0    70.9       0       0     0.0      71      71     1.9   0.71   2.49
32 MAKEUP CARBON COST                 50.0       0       0       0   346.1       0       0     0.0     346     346     1.9   0.69   2.42

33 REVERSE OSMOSIS                     1.0     938    1154    2530    94.7    1949       0    56.4     228     337    92.3  33.70 117.95
33 REVERSE OSMOSIS                    10.0    6680    8216   24102   733.9    2647       0   564.3    1856    2632    72.1  26.32  92.12
33 REVERSE OSMOSIS                    50.0   28986   35653  116486  3436.4    4950       0  2821.5    8803   12160    66.7  24.34  85.13

34 OZONE GENERATION SYSTEM             1.0     385     474     316     2.8     531       0     0.0      21      66    18.1   6.60  23.10
34 OZONE GENERATION SYSTEM            10.0    1719    2114    2485    13.4    1371       0     0.0     127     327     9.0   3.27  11.45
34 OZONE GENERATION SYSTEM            50.0    6268    7710   11045    54.3    5868       0     0.0     555    1283     7.0   2.57   8.98

35 OZONE CONTACT CHAMBER               1.0      14      17       0     0.0       0       0     0.0       0       2     0.5   0.20   0.70
35 OZONE CONTACT CHAMBER              10.0      74      91       0     0.0       0       0     0.0       0       9     0.2   0.09   0.32
35 OZONE CONTACT CHAMBER              50.0     207     255       0     0.0       0       0     0.0       0      24     0.1   0.05   0.17

36 PLANT ADMINISTRATION                1.0       0       0       0     2.5     100       0     0.0       4       4     1.1   0.40   1.40
36 PLANT ADMINISTRATION               10.0       0       0       0     6.9     464       0     0.0      12      12     0.3   0.12   0.42
36 PLANT ADMINISTRATION               50.0       0       0       0    17.2    1291       0     0.0      30      30     0.2   0.06   0.21

37 YARD MAINTENANCE                    1.0       0       0       0     1.2     691       0     0.0       8       8     2.2   0.80   2.80
37 YARD MAINTENANCE                   10.0       0       0       0     4.8    1866       0     0.0      23      23     0.6   0.23   0.80
37 YARD MAINTENANCE                   50.0       0       0       0    12.3    2657       0     0.0      39      39     0.2   0.08   0.27

          TOTAL COST (INCLUDES YARD PIPING AT 14.0 %) FOR COMPONENTS   1 THRU  37:   LEVEL 11 TREATMENT

LEVEL 11 TREATMENT                     1.0    5836    7185    4049   209.1   16749   15770   750.3     666    1328   360.6 134.54 470.90
LEVEL 11 TREATMENT                    10.0   28955   35614   33701   962.6   41829   12073  7502.6    3577    6926   190.2  69.42 242.98
LEVEL 11 TREATMENT                    50.0  107051  131674  160594  4292.9  130836   49436 37513.2   16231   28640   157.0  57.32 200.61


         ***** TREATMENT SYSTEM COST SUMMARIES INCLUDING YARD PIPING AT 14.0 PERCENT OF CONSTRUCTION COST *****

LEVEL 11 TREATMENT PLANT               1.0    5836    7185    4049   209.1   16749   15770   750.3     666    1328   360.6 134.54 470.90
LEVEL 11 TREATMENT PLANT              10.0   28955   35616   33701   962.6   41829   12073  7502.6    3577    6926   190.2  69.42 242.98
LEVEL 11 TREATMENT PLANT              50.0  107051  131674  160594  4292.9  130836   49436 37513.2   16231   28640   157.0  57.32 200.61
```

Figure 5. SAMPLE COMPUTER OUTPUT

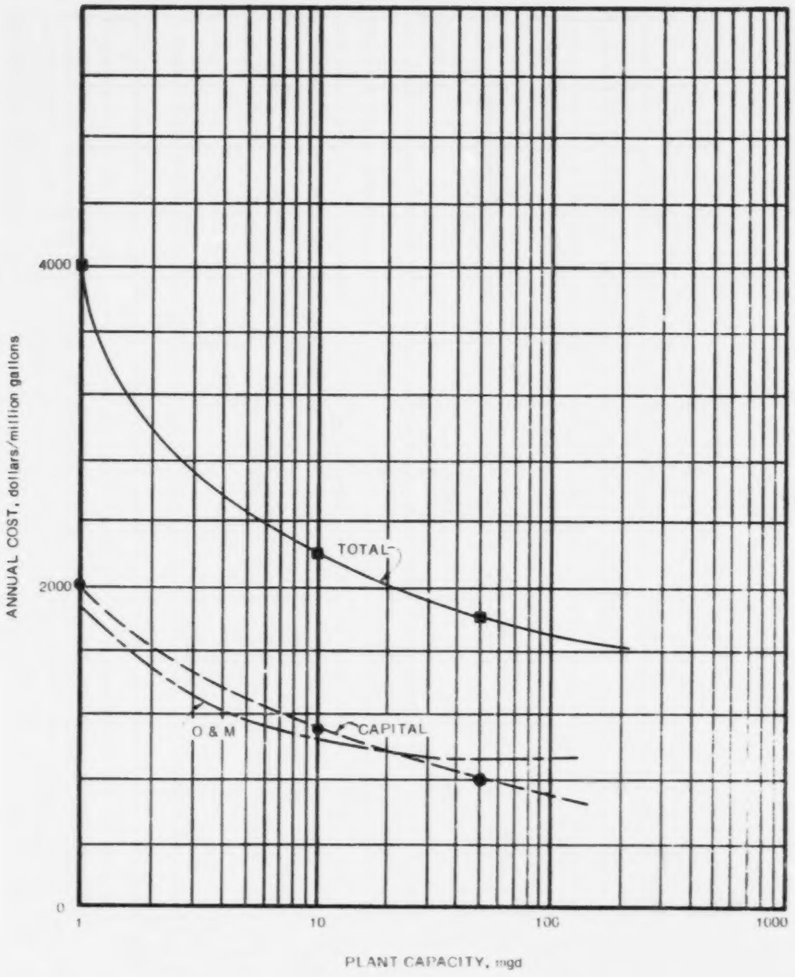


Figure 6

ANNUAL COSTS FOR LEVEL II TREATMENT

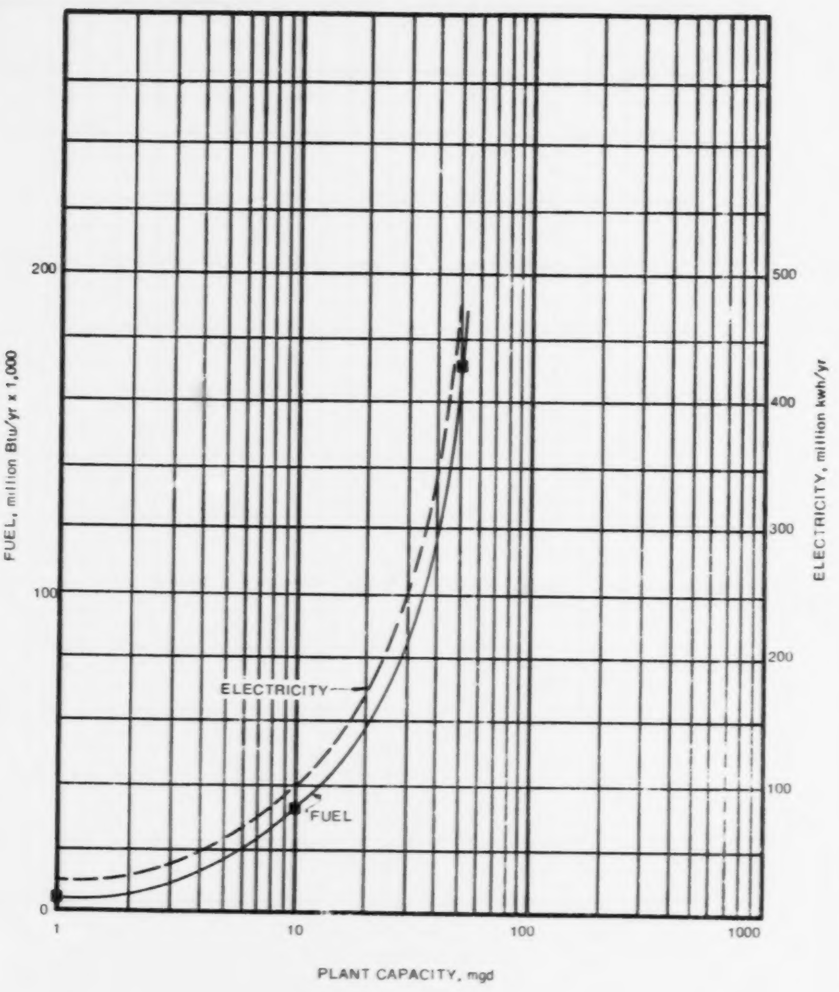


Figure 7

ENERGY AND FUEL REQUIREMENTS FOR LEVEL II TREATMENT

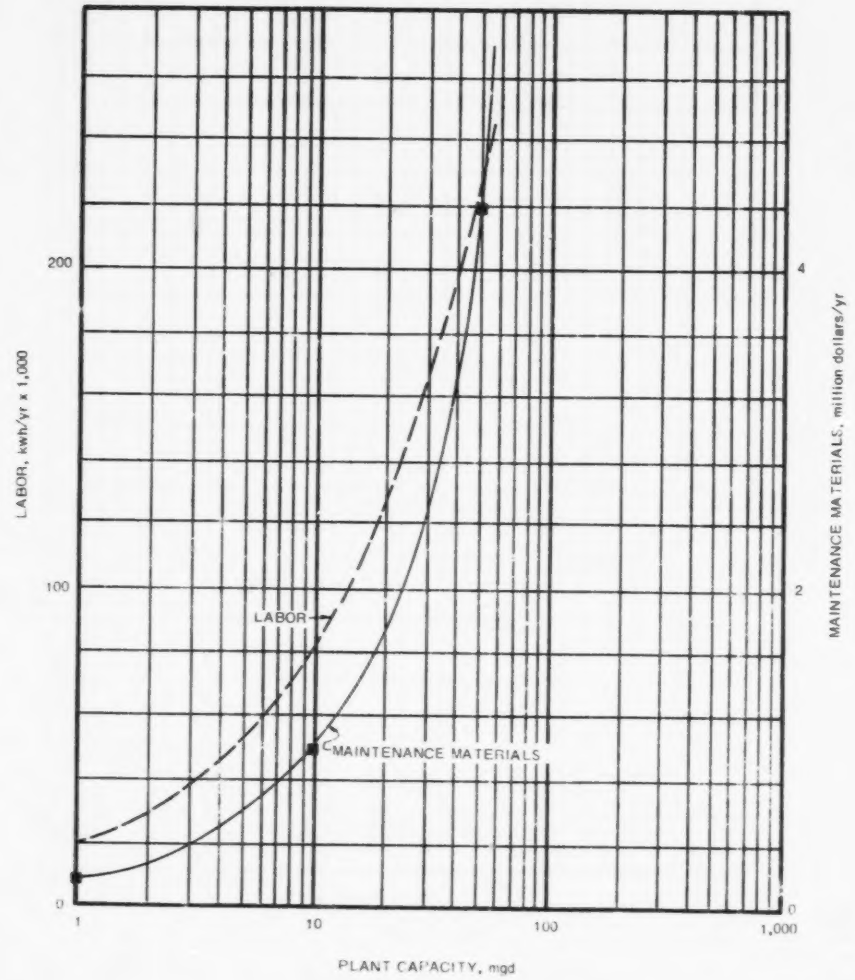


Figure 8

O & M REQUIREMENTS FOR LEVEL II TREATMENT

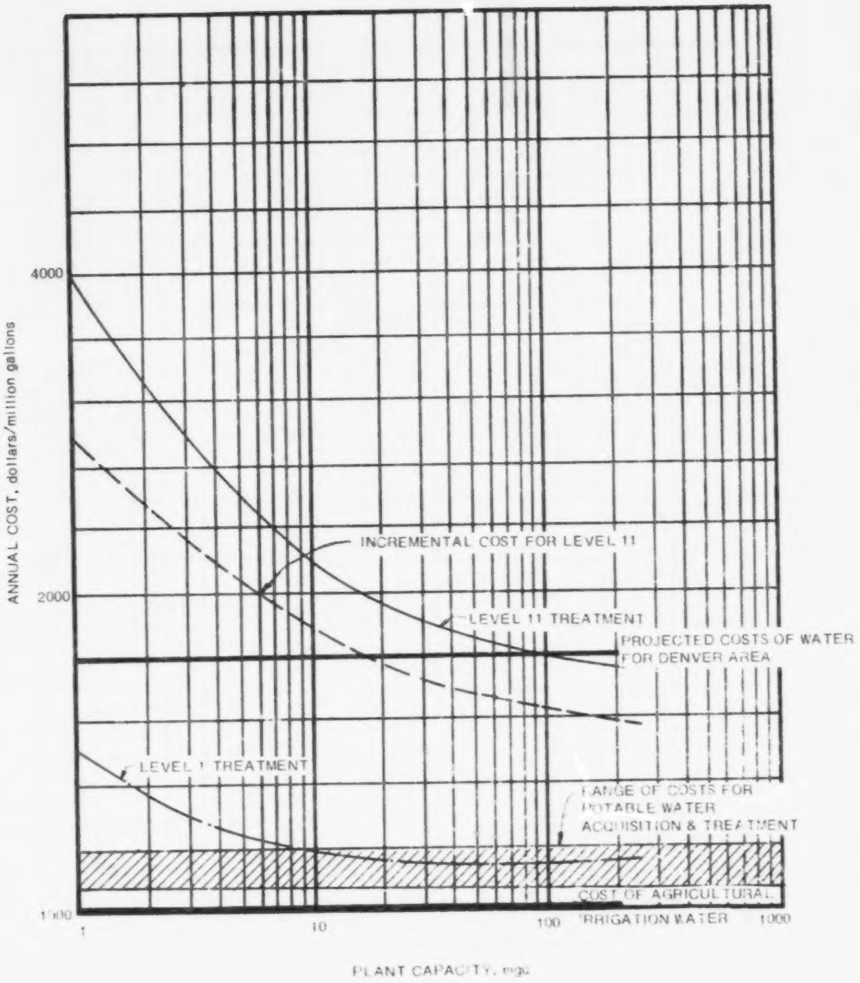


Figure 9

COMPARISON OF TREATMENT FOR REUSE AND RAW WATER SUPPLIES

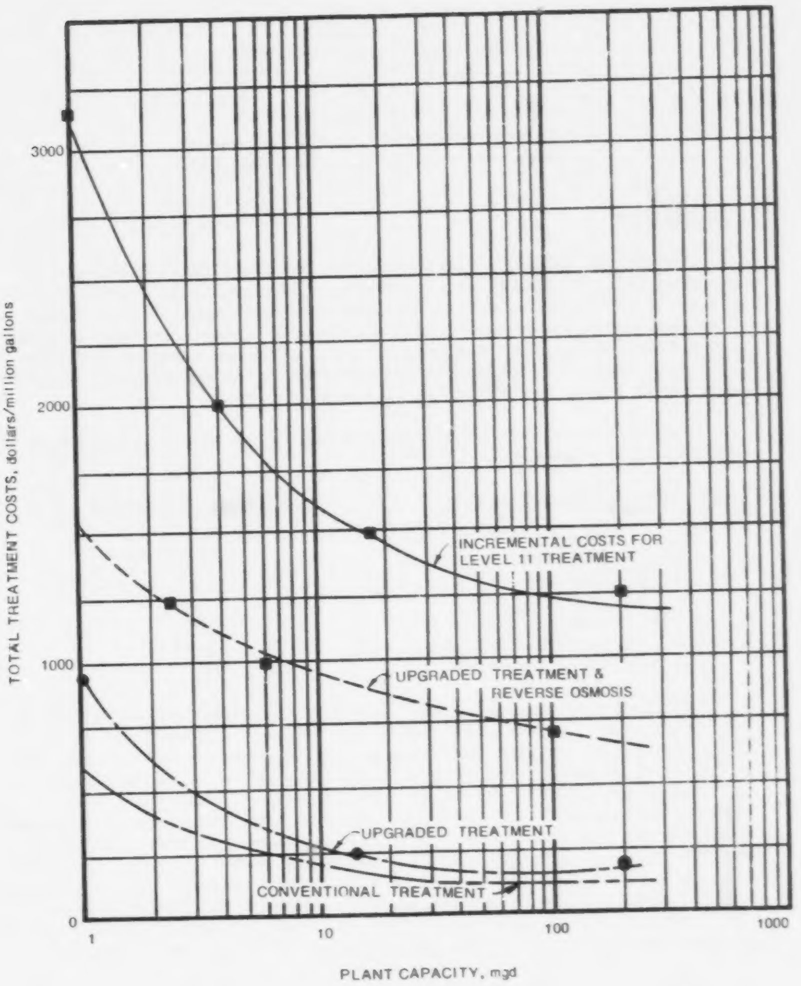


Figure 10
WATER TREATMENT COSTS

FEASIBILITY OF TREATING HOSPITAL WASTEWATER BY LIME AND PRESSURE OZONATION

A. G. Hill
J. B. Howell
Louisiana Tech University
Ruston, Louisiana 71272

J. J. McCarthy
U.S. Army Environmental Hygiene Agency
Aberdeen Proving Grounds, Maryland 21010

INTRODUCTION

Due to the nature of their mission, military field hospitals require greater per capita water resources than similarly manned combat units. With the introduction of modernized equipment a trend toward increased water usage may be anticipated. In order to enhance the deployability of hospital units in arid environments research has been conducted examining the feasibility of renovating non-sanitary wastewater to a quality suitable for sub-potable uses.

The following list identities sources of wastewater expected to be available for treatment[1]:

Effluent Source	Approximate TOC (mg/L)
Shower	34
Operating Room	225
Kitchen	1120
Laboratory	246
X-Ray	800
Colorfast Laundry	605
Woolens Laundry	236

The U.S. Army Medical Research and Development Command has set a tentative water quality goal for trace organics at 5 mg/L TOC or 10 mg/L COD in the finished water being considered for reuse. A previous study[2] evaluated the treatment sequence consisting of ultrafiltration, reverse osmosis, and ultraviolet light activated ozonation. When treated individually, all hospital waste streams except for the laboratory and x-ray effluents met the organics limitation after the reverse osmosis step. Bench scale studies had shown that laboratory and x-ray reverse osmosis permeates could be effectively oxidized by ozone aided by ultraviolet light. However, due to equipment and design problems, this was not demonstrated during the pilot scale tests. These workers[2] recommended further pilot scale evaluation of ozone contacting.

The objective of the present study was to evaluate the feasibility of contacting ozone with laboratory waste reverse osmosis permeate in a tall bubble column operated at elevated pressure. Ozone/gas mixtures are not generally applied at pressures in excess of about 200 kPa when O_3 is supplied directly from the ozone generator. This is due to the adverse effect of pressure on the electric discharge method of ozone formation. In this investigation ozone/gas mixture was compressed in a two stage process following the O_3 generator. The effects of changes in temperature and pH (through addition of $Ca(OH)_2$) were also observed.

EXPERIMENTAL

Ozone was generated from either dry air or oxygen. A Welsbach Model T-23 ozonizer was used in all runs for ozone production. When O_3 was generated from O_2 the oxygen gas was supplied directly to the ozonizer from the O_2 gas cylinder which had been obtained from a welding supply house. When ozone was generated from air an OREC Model 03V92-AR ozonizer was used for air drying only (in an air conditioned room). The dry air from this unit was fed through a 10 m length of rubber tubing to the Welsbach unit for ozone generation.

Development of the compression process has been described in detail previously[3]. This consisted of two Cast Model PAB-10 separate drive, oil-less piston compressors, operated in series. Modified piston rings were installed in the compressors to enable them to operate at low RPM (maximum 300 RPM) and maintain a tight gas seal.

A line drawing of the 7.7 m tall bubble column is given in Figure 1. The majority of the wetted portion of the reactor was constructed from jacketed sections of 5.25 cm inside diameter stainless steel pipe (Schedule 40). The two lowest sections were each 2.44 m in length. The top section of the column was made transparent by adaptation of a used, nominal 5 cm diameter, stainless steel rotameter. The original rotameter glass was replaced with a 1.22 m length of heavy wall pyrex glass (6.35 cm outside diameter). The column sections were connected by flanged joints using teflon gaskets (3.2 mm thick).

Liquid sample ports were located at the reactor base, 2.29 m (Level 2), 4.73 m (Level 3), and 5.95 m (Level 4). The gas sparger consisted of a porous kellundite (ceramically bonded alumina) disk 33.3 mm in diameter, 6.3 mm thick, 50 μm nominal partical retention (Ferro Corporation). This disk was mounted in two machined pieces of stainless steel, held together by a set screw. The sparger assembly was attached to a tube fitting that entered through the center of the bottom blind flange.

A liquid recycle line (6 mm stainless steel tubing) was used during all tests. This included a centrifugal pump to circulate a flow of 60 L/h (the reactor held about 15 L). Liquid drawn off at the bottom was reintroduced at the top of the column.

The cooling/heating jackets in the three lowest column sections were fed a continuous flow of thermostated water originating in a constant temperature bath. The jackets were connected in sequence from bottom to top with the liquid overflow returning to the bath.

Dissolved ozone was measured by a method similar to Wynn, *et al.*[4] This included placing 40 mL of 10 percent by weight KI solution and five drops of five percent by weight KOH into a 250 mL graduated cylinder. After clearing the sample line, liquid from the column was drawn to the 240 mL mark. This solution was then acidified with 10 mL of 1.0 M H_2SO_4 and titrated with 0.02 N $Na_2S_2O_3$. This procedure was subject to some error due to entrainment of gas bubbles and interference from oxidizing intermediates formed during reaction.

Ozone concentration in the gas phase was determined several times during each run using the off gas from the top of the column, (EXGAS). At the end of a run the ozone/gas mixture leaving the compressors was measured at the same operating pressure used during the run. Duplicate measurements were made on this stream (INGAS). Ozone in the gas phase was determined by sparging the entire gas flow into 600 mL of 2.5 percent KI solution. This solution was then acidified with 15 mL 1.0 M H_2SO_4 and titrated with 0.1 N $Na_2S_2O_3$. A gas sample of about 2.8 L was used, as determined by a wet test meter downstream of the KI absorber. Gas flow rate was calculated by recording the time required to draw this sample. Ozone concentrations and gas flow rates are reported in values adjusted to 20°C and 101 kPa. A factor was applied to account for the moisture pickup during analysis. This allowed the gas concentrations and flow rates to be given on a dry basis.

Chemical oxygen demand (COD) was determined by the procedure given in Standard Methods[5]. Reagent concentrations were twice as strong as those listed in the "Alternate Procedure for Dilute Samples." A Coleman Model 38A pH meter was used to determine pH values.

The laboratory waste reverse osmosis permeate was simulated by using the synthetic formulary[6] given in Table 1. This mixture had a theoretical COD = 535 mg/L. Lime as $Ca(OH)_2$ was added to 24 out of a total of 35 runs with the laboratory waste. The lime dosage and column operating conditions are summarized for each run in Table 2. Acetone was deleted from the laboratory waste blend during runs LNA1 and LNA2.

Operating pressure was programmed during two runs. In run L8 the pressure was maintained at 184 kPa for four hours and then was raised to 791 kPa for an additional five hours. Run LNA1 was performed at 177 kPa for three hours and then 791 kPa for the next six hours. The ozone/gas mixture was compressed in a single stage when the column was operated at pressures ranging from 170 to 184 kPa (measured in the sparger feed gas line). Two stage compression was used with column pressures ranging from 446 to 791 kPa.

All runs were performed in a semi-batch mode. Upon filling the column with liquid the recycle pump was started and the level was recorded. The carrier gas flow train was then begun. After the column had reached operating pressure another level measurement was recorded. These values were used to compute gas holdup. Zero time was taken at the time of initiation of ozonizer power. The first COD sample was drawn after the aerated level was recorded and before the initiation of ozonizer power. About 150 mL of liquid was taken for each COD sample. This was placed in a 250 mL bottle. The liquid sample was sparged with room air for about 30 s using an aquarium pump in order to remove dissolved O_3. When pH measurements were made they were taken on the same samples used for COD. Analyses of the COD samples were made within 24 hours of the run.

There was a tendency to put less liquid in the column during the low pressure tests. This was done to allow for the greater expansion of the liquid phase after the start of bubbling. Semi-batch ozone application rates were computed from the product of the INGAS concentration and flow rate divided by the initial liquid volume. EXGAS/INGAS flow ratio was computed from the average EXGAS and INGAS flow rates.

RESULTS AND DISCUSSION

Loss of ozone during compression may be attributed to either decomposition or gas leakage. The efficiency of O_3 compression may be observed by comparing the values of the INGAS O_3 concentration and flow rate for a series of experiments that had identical ozonator settings. Ozone was generated at identical conditions during the following series of runs:

Feed Gas	Average O_3 Production (g/h)	Runs
Air	2.21	LL7 - LL15
O_2	6.58	L4 - L9, LNA1, LNA2, LL1, LL2
O_2	6.43	LL16 - LL24
O_2	4.58	L1 - L3, LL3 - LL6

Starting solution for the first six runs with the laboratory waste (L1 - L6) was prepared by weighing an approximate 190 L volume of dilution water contained in a polyethylene tank. The organic components were then added to give the prescribed concentrations. The initial COD value was expected to be about 535 mg/L. However, over the course of the four days required to make these tests, the starting solution deteriorated in strength. By run L6 the initial COD had dropped to 280 mg/L. Biological oxidation was suspected. All subsequent tests were performed with fresh solution prepared by diluting a laboratory waste concentrate. The mixture listed in Table 1 was obtained when 100 mL of laboratory waste concentrate was diluted to 19 L.

The mass balance results are summarized in Table 3.

The COD goal of 10 mg/L was not reached during the first nine runs (L1 - L9) with the laboratory waste. In these tests the highest ozone application ratio occurred during runs L8 and L9 where an O_3 applied/COD_I value of about eight to one was accumulated over nine hours. The following trends were observed during this series of experiments: pH changed in an erratic manner, generally in a narrow range; dissolved ozone stabilization accompanied the appearance of a resistant residual COD, resulting in large values of EXGAS ozone concentration near the end of each run; the oxidized COD was removed with an apparent stoichiometry close to the theoretical value of three to one.

The effect of elevated pressure was difficult to determine during runs L1 through L9. A comparison of the final COD values would indicate that better results were obtained at low pressure. However, this may have been due to greater ozone compression survival and the use of reduced initial liquid volumes during low pressure runs. Elevated pressure appears to have had a more nearly neutral effect when COD values are compared after a three to one ratio in O_3 applied/COD_I.

Acetone was deleted from the laboratory waste mixture during runs LNA1 and LNA2. At the end of these tests, the COD was reduced to less than 1.0 mg/L. This would indicate that either acetone, or a decomposition product of acetone, was responsible for the refractory residual COD that occurred in the previous runs. The ratio of ozone absorbed to COD_I was relatively high during runs LNA1 and LNA2. However, this does not reflect a loss in efficiency during COD oxidation. In both runs the off gas ozone concentration gradually decreased after the COD had been completely oxidized. This loss in stabilization of dissolved ozone may have been due to stripping of the inorganic carbon formed during oxidation.

Lime as $Ca(OH)_2$ was added to the last 24 runs with the laboratory waste. During runs LL1 and LL2 the lime dosage was 423 mg/L. In these runs the final COD values were respectively 1.2 and 0.4 mg/L. There appeared to be little difference in the high and low pressure runs at this dosage of lime.

The lime dosage was varied during four high pressure runs (LL3 - LL6), with the objective being to find the minimum required dose. In this series of tests approximately the same performance was obtained with doses of 317, 211, and 106 mg/L $Ca(OH)_2$. In run LL6 a small amount of precipitate was observed in the transparent section of the column. Although no lime was added to the initial solution, it is believed that there was some $Ca(OH)_2$ or $CaCO_3$ left in the column from previous runs, and this trace amount elevated the pH to about nine at the start of this run. The final COD was above 20 mg/L at the end of run LL6. As a result of these tests the decision was made to use a dosage of 106 mg/L $Ca(OH)_2$ in all remaining runs with the laboratory waste.

Ozone was generated from air during runs LL7 through LL15. In these tests the laboratory waste/lime mixture was treated in an experimental matrix consisting of three temperatures and three pressures. Low final COD's were obtained at operating temperatures of 30 and 40°C. However, at 20°C a resistant residual COD occurred, in excess of the 10 mg/L goal. Smaller final COD values were found during the low pressure runs at each operating temperature. However, the difference in percent COD reduction due to pressure, was small when compared to the variation in the ratio of O_3 applied to COD_I.

Ozone was generated from oxygen during the last nine runs with the laboratory waste/lime mixture. The use of oxygen as a synthesis gas, and small changes in the ozonator settings, allowed the duration of these runs to be reduced to 6.0 h. The effects of variation in temperature and pressure observed during runs LL16 through LL24, were similar to those found in previous runs when O_3/air was used as the feed gas.

There was a trend toward lower pH values with time, in all runs using the laboratory waste/lime mixture. When the lime dosage was 106 mg/L, the initial pH value of between ten to 12, fell to a final value ranging from six to nine. This reduction in pH can be attributed to the formation of inorganic carbon as a product of reaction.

Large values of dissolved ozone and EXGAS ozone concentration were observed toward the end of each run in the first series of tests (L1 - L9) with the laboratory waste mixture. The latter few COD values in runs L7 and L8 asymptotically approached resistant plateau concentrations. When lime was added to the laboratory waste, the onset of high EXGAS ozone concentration was delayed until the COD had been reduced to a somewhat lower value. However, an important part of the oxidation did occur after "reaction rate limited" conditions had ensued. COD values less than 10 mg/L were only obtained after at least 1.0 h reaction time at peak dissolved O_3 concentration during the following 12 runs: LL1, LL2, LL7, LL8, LL9, LL10, LL11, LL19, LL20, LL22, LL23, and LL24. The average value of the ratio of O_3 absorbed to COD_I was 3.87 for these 12 runs.

The data that has been summarized in Table 3 was computed for each run after graphically illustrating the raw data. Some samples of the graphical results are given in Figures 2 through 4 for runs L8, LL7, and LL9.

CONCLUSIONS

The laboratory waste reverse osmosis permeate was effectively oxidized to a COD value below 10 mg/L during these experiments. Addition of lime proved to be a more important factor than elevation in pressure. However, economical use of ozone requires consideration of the feasibility of applying oxygen recycle in the ozone generating flow scheme. It is likely that the potential advantages of pressure ozonation will become more important when considering this possibility.

ACKNOWLEDGEMENT

The experimental work described here was performed under Contract DAMD 17-77-C-7030 with the U.S. Army Medical Research and Development Command.

REFERENCES

1. McNULTY, K. J., GOLDSMITH, R. L., AND GOLLAN, A. Z., Post Treat-Treatment of MUST Hospital Wastewaters by Ozonation for Reuse, Water-1976: I. Physical, Chemical Wastewater Treatment (G. F. Bennett, editor) AIChE Symposium Series No. 166, Vol 73, (1977).

2. GOLLAN, A. Z., McNULTY, K. J., GOLDSMITH, R. L., KLEPER, M. H., and GRANT, D. C., Evaluation of Membrane Separation Processes, Carbon Adsorption, and Ozonation for Treatment of MUST Hospital Wastes, Preliminary Draft Final Report to U.S. Army Medical Research and Development Command, Contract DAMD 17-74-C-4066 (1975).

3. HILL, A. G., and HOWELL, J. B., Compression of O_3/O_2 and O_3/Air Gas Mixtures, Proc. Symp. on Advanced Ozone Technology, International Ozone Institute, Toronto, (Nov. 1977).

4. WYNN, C. S., KIRK, B. S., and McNABNEY, R., Pilot Plant for Tertiary Treatment of Wastewater With Ozone, Environmental Protection Technology Series, EPA-R2-73-146, (Jan. 1973), NTIS Order No. PB-219 877.

5. Standard Methods for the Examination of Water and Wastewater, AWWA, APHA, WPCF, 13th Edition, pp. 495-499, (1971).

6. McCARTHY, J. J., COWEN, W. F., CHIAN, E. S. K., and PETERMAN, B. W., Evaluation of an Air Stripping - Ozone Contactor System, Technical Report 7707, U.S. Army Medical Bioengineering Research and Development Laboratory, (1977).

Table 1

Synthetic Laboratory Waste

(Pretreated by Ultrafiltration and Reverse Osmosis)

Component	Concentration (mg/L)	Theoretical COD (mg/L)	% of Total Theoretical COD
Methanol	289.1	433.7	81.09
Acetone	32.8	72.4	13.54
Formaldehyde	10.6	11.3	2.11
Ethanol	3.3	6.9	1.29
Diethyl Ether	1.8	4.7	0.88
Glycerol	2.1	2.6	0.49
O-phenyl phenol	0.4	1.1	0.21
Urea	0.9	0.7	0.13
O-benzyl-p-chlorophenol	0.3	0.7	0.13
Xylenol	0.2	0.5	0.09
Isopropanol	0.1	0.2	0.04
Total		534.8	

Table 2

Operating Conditions

Run	$Ca(OH)_2$ Dose (mg/L)	Column Temp. (°C)	Column Press. (kPa)	INGAS Conc. (mg/L)	INGAS Flow (SLPM)	Initial Liquid (L)	Semi-Batch O_3 App. Rate |mg/(L•h)|	Duration (h)	EXGAS/INGAS Flow
L1	0	30	791	25.1	2.62	15.4	255.7	5	0.981
L2	0	30	446	26.1	2.66	15.3	276.8	5	1.083
L3	0	30	170	26.8	2,82	14.4	314.9	5	0.972
L4	0	30	791	25.1	3.17	15.2	314.6	5	0.997
L5	0	30	446	23.9	3.29	15.3	308.0	5	0.973
L6	0	30	184	29,5	3.29	14.0	416.7	5	0.930
L7	0	40	177	34.8	3.26	14.5	470.7	6	0.908
L8	0	30	184/791	34.1	3.21	14.4	454.9	9	0.944
L9	0	30	184	34.5	3.25	14.9	452.1	9	0.938
LNA1	0	30	177/791	33.8	3.20	14.7	442,9	9	0.953
LNA2	0	30	170	34.7	3.27	14.5	468.4	9	0.911
LL1	423	30	791	35.0	3.10	15.0	434.6	9	0.984
LL2	423	30	184	36.4	3.17	14,5	477.8	9	0.946
LL3	317	30	791	24.7	2.80	15.1	275.1	7.5	0.957
LL4	211	30	791	23.2	2.82	15.0	261.6	7.5	0.961
LL5	106	30	791	24.2	2.88	14.5	288.0	7.5	0.962
LL6	Trace	30	791	25.2	2.75	15.2	272.4	7.5	0.964

Table 2 (continued)

Run	$Ca(OH)_2$ Dose (mg/L)	Column		INGAS		Initial Liquid (L)	Semi-Batch O_3 App. Rate $\lvert mg/(L\cdot h)\rvert$	Duration (h)	$\frac{\text{EXGAS}}{\text{INGAS}}$ Flow
		Temp. (°C)	Press. (kPa)	Conc. (mg/L)	Flow (SLPM)				
LL7	106	30	184	11.1	2.98	14.2	139.2	14	0.990
LL8	106	30	446	11.7	2.80	14.9	131.9	14	1.050
LL9	106	30	791	12.4	2.93	15.3	142.1	14	0.945
LL10	106	40	184	12.8	3.05	14.7	158.8	13	0.898
LL11	106	40	446	13.0	2.97	15.0	153.9	13	0.987
LL12	106	40	791	12.8	2.58	14.8	133.3	13	0.946
LL13	106	20	170	12.5	3.07	14.8	154.6	13	0.879
LL14	106	20	446	12.3	2.85	15.0	139.6	13	0.930
LL15	106	20	791	12.7	2.42	15.4	119.6	13	0.946
LL16	106	20	170	29.6	3.58	14.8	428.9	6	0.866
LL17	106	20	446	28.9	3.32	15.0	383.9	6	0.943
LL18	106	20	791	29.4	2.92	15.1	340.0	6	0.890
LL19	106	30	177	29.0	3.59	15.0	417.5	6	0.905
LL20	106	30	446	30.7	3.38	15.2	408.9	6	0.956
LL21	106	30	791	31.0	2.92	15.0	362.0	6	0.952
LL22	106	40	184	31.6	3.52	15.0	444.9	6	0.920
LL23	106	40	446	30.9	3.39	15.3	412.3	6	0.971
LL24	106	40	791	30.0	3.31	15.4	385.9	6	0.873

Table 3

Mass Balance Results

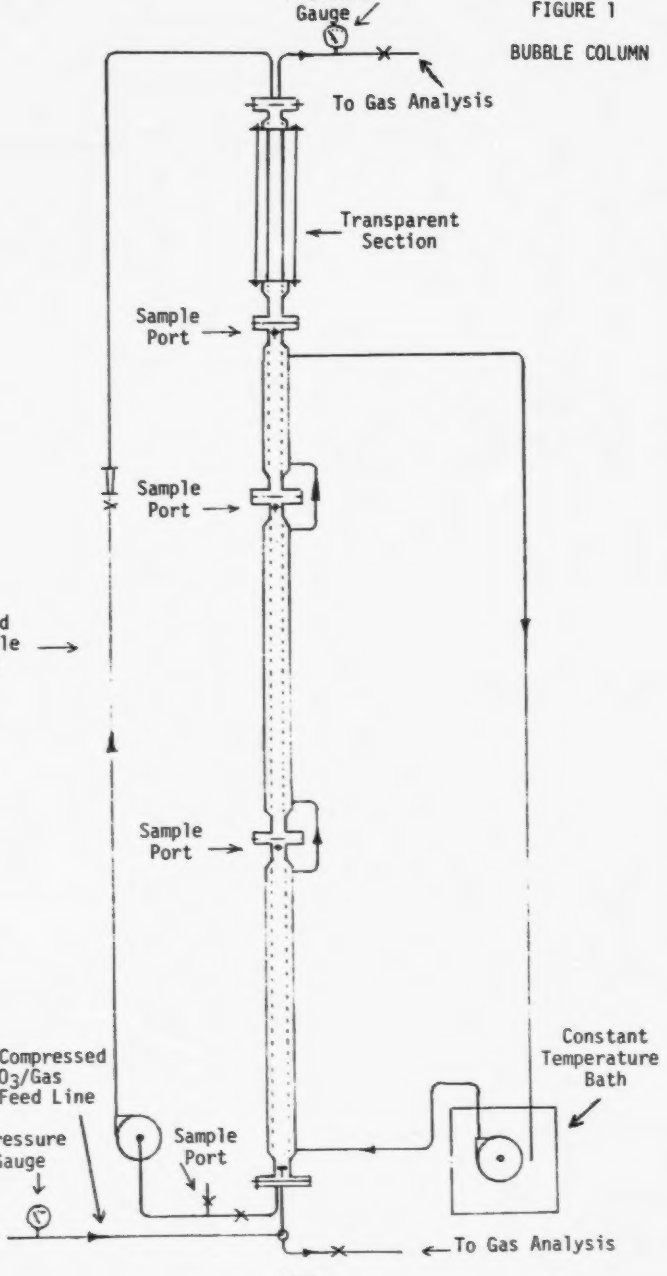
Run	COD_I (mg/L)	COD After 3:1 O_3 App./COD_I (mg/L)	Final O_3 App./COD_I (mg/mg)	Final COD (mg/L)	COD Reduction (%)	O_3 Abs./O_3 App. (mg/mg)	O_3 Abs./COD_I (mg/mg)
L1	408.4	80.2	3.13	71.7	82.4	.843	2.64
L2	410.9	71.9	3.37	57.7	86.0	.746	2.51
L3	402.8	79.5	3.91	45.9	88.6	.655	2.56
L4	370.9	76.0	4.24	59.4	84.0	.743	3.15
L5	330.3	78.5	4.66	45.6	86.2	.574	2.68
L6	280.0	115.8	7.44	26.4	90.6	.457	3.40
L7	472.0	126.6	5.98	30.0	93.6	.649	3.88
L8	484.8	148.1	8.44	33.6	93.1	.444	3.75
L9	534.4	109.5	7.61	20.0	96.3	.386	2.94
LNA1	422.4	98.1	9.45	0.4	99.9	.489	4.61
LNA2	447.0	113.2	9.43	0	100	.609	5.74
LL1	510.4	89.4	6.81	1.2	99.8	.675	4.60
LL2	488.0	84.3	7.83	0.4	99.9	.548	4.29
LL3	523.6	47.2	3.94	12.7	97.6	.760	2.99
LL4	493.6	46.1	3.97	12.7	97.4	.775	3.08
LL5	474.3	61.0	4.55	13.8	97.1	.753	3.43
LL6	502.8	61.2	4.06	20.2	96.0	.826	3.36

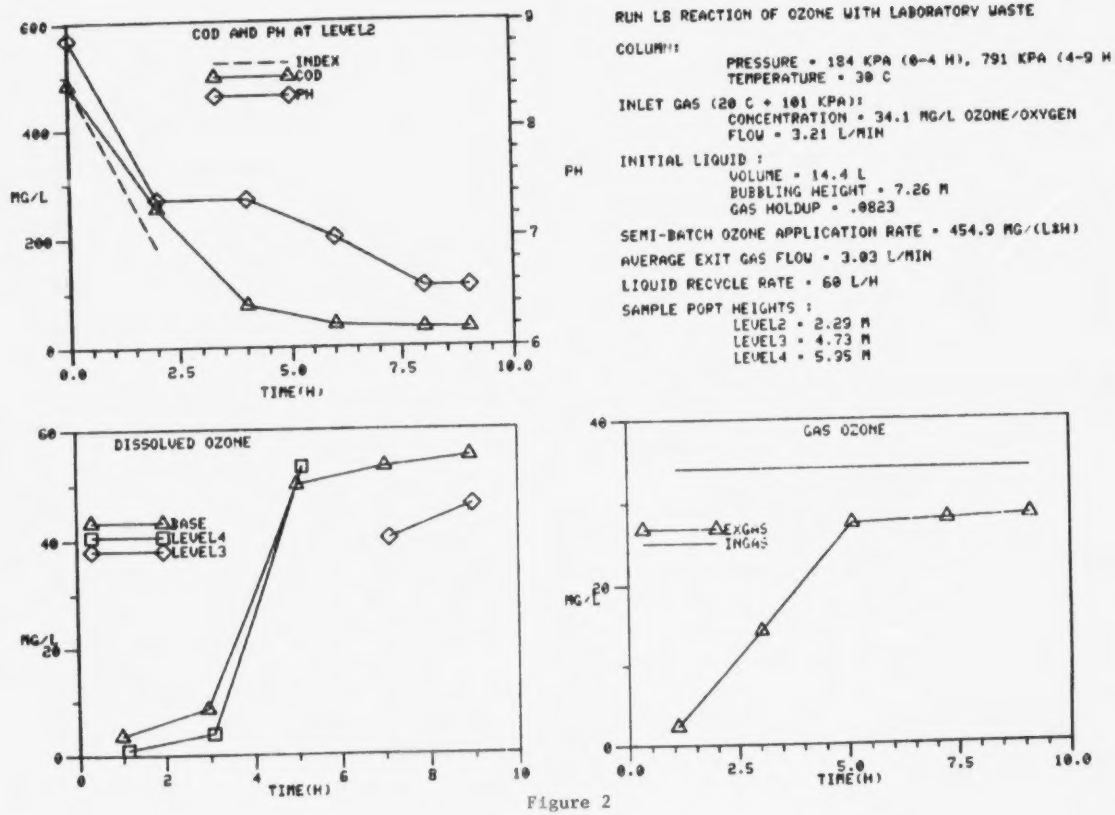
Table 3 (continued)

Run	COD_I (mg/L)	COD After 3:1 O_3 App./COD_I (mg/L)	Final O_3 App./COD_I (mg/mg)	Final COD (mg/L)	COD Reduction (%)	$\frac{O_3\ \text{Abs.}}{O_3\ \text{App.}}$ (mg/mg)	$\frac{O_3\ \text{Abs.}}{COD_I}$ (mg/mg)
LL7	417.4	53.5	4.67	2.0	99.5	.642	3.00
LL8	264.0	78.5	6.99	(-0.8)	100	.702	4.91
LL9	478.6	40.1	4.16	7.5	98.4	.829	3.45
LL10	484.9	51.1	4.26	0.8	99.8	.779	3.32
LL11	495.2	37.7	4.04	0.6	99.9	.814	3.29
LL12	515.7	36.9	3.36	13.6	97.4	.918	3.08
LL13	417.4	70.3	4.82	19.1	95.4	.673	3.24
LL14	439.4	86.3	4.13	35.7	91.9	.775	3.20
LL15	486.3	76.0	3.19	60.0	87.7	.926	2.96
LL16	511.6	84.1	5.03	24.5	95.2	.605	3.04
LL17	490.2	89.1	4.70	44.5	90.9	.713	3.35
LL18	509.0	89.3	4.01	43.1	91.5	.831	3.33
LL19	510.4	62.7	4.91	1.4	99.7	.657	3.22
LL20	441.7	98.4	5.55	6.3	98.6	.709	3.94
LL21	507.4	64.3	4.28	29.9	94.1	.804	3.44
LL22	489.4	71.3	5.45	2.3	99.5	.709	3.87
LL23	483.0	79.2	5.12	(-0.1)	100	.802	4.11
LL24	437.1	116.3	5.30	6.7	98.5	.847	4.49

FIGURE 1

BUBBLE COLUMN





RUN L8 REACTION OF OZONE WITH LABORATORY WASTE

COLUMN:
PRESSURE = 184 KPA (0-4 H), 791 KPA (4-9 H)
TEMPERATURE = 30 C

INLET GAS (20 C + 101 KPA):
CONCENTRATION = 34.1 MG/L OZONE/OXYGEN
FLOW = 3.21 L/MIN

INITIAL LIQUID :
VOLUME = 14.4 L
BUBBLING HEIGHT = 7.26 M
GAS HOLDUP = .0823

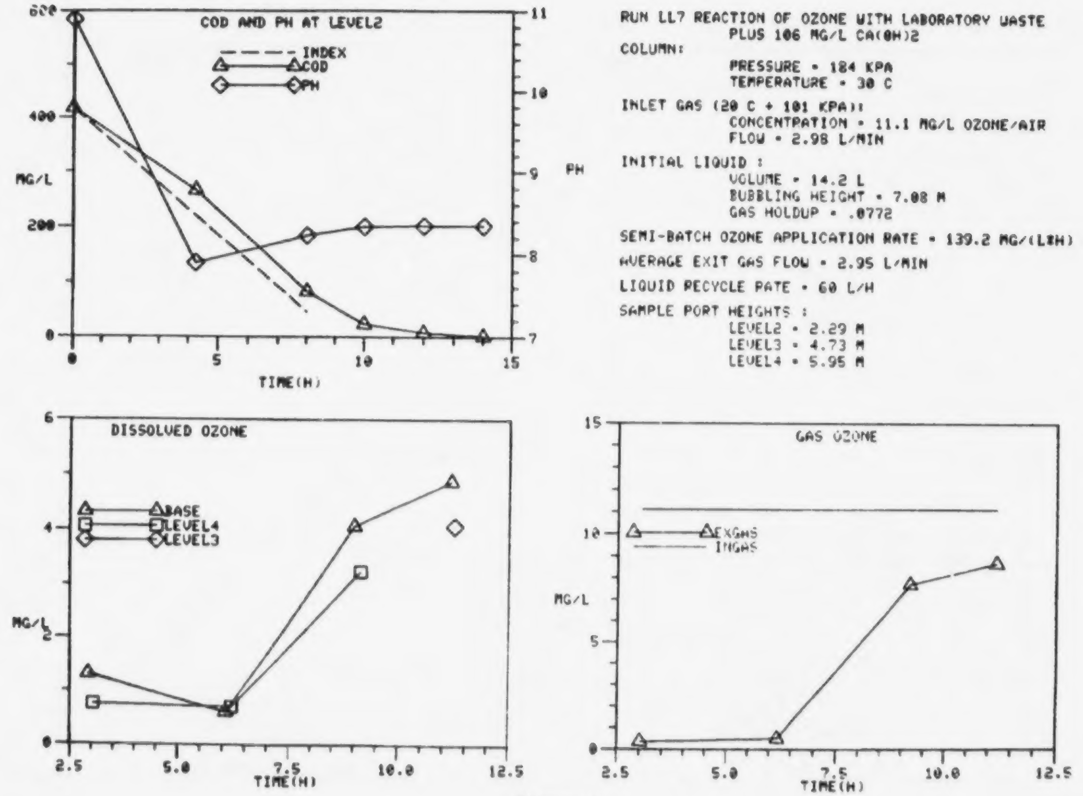
SEMI-BATCH OZONE APPLICATION RATE = 454.9 MG/(L*H)

AVERAGE EXIT GAS FLOW = 3.03 L/MIN

LIQUID RECYCLE RATE = 60 L/H

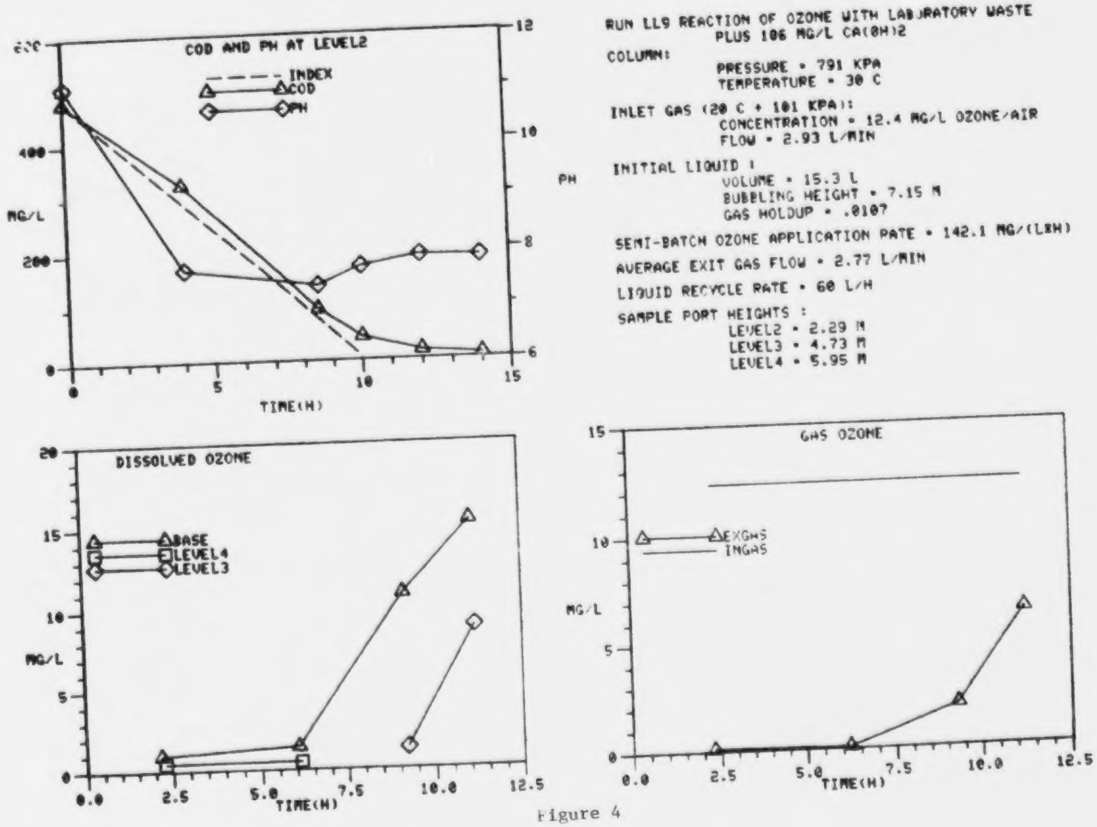
SAMPLE PORT HEIGHTS :
LEVEL2 = 2.29 M
LEVEL3 = 4.73 M
LEVEL4 = 5.95 M

Figure 2



RUN LL7 REACTION OF OZONE WITH LABORATORY WASTE
PLUS 106 MG/L CA(0H)2
COLUMN:
PRESSURE = 184 KPA
TEMPERATURE = 30 C
INLET GAS (20 C + 101 KPA):
CONCENTRATION = 11.1 MG/L OZONE/AIR
FLOW = 2.98 L/MIN
INITIAL LIQUID :
VOLUME = 14.2 L
BUBBLING HEIGHT = 7.08 M
GAS HOLDUP = .0772
SEMI-BATCH OZONE APPLICATION RATE = 139.2 MG/(L*H)
AVERAGE EXIT GAS FLOW = 2.95 L/MIN
LIQUID RECYCLE RATE = 60 L/H
SAMPLE PORT HEIGHTS :
LEVEL2 = 2.29 M
LEVEL3 = 4.73 M
LEVEL4 = 5.95 M

Figure 3



RUN LL9 REACTION OF OZONE WITH LABORATORY WASTE
PLUS 106 MG/L CA(OH)2

COLUMN:
PRESSURE = 791 KPA
TEMPERATURE = 30 C

INLET GAS (20 C + 101 KPA):
CONCENTRATION = 12.4 MG/L OZONE/AIR
FLOW = 2.93 L/MIN

INITIAL LIQUID :
VOLUME = 15.3 L
BUBBLING HEIGHT = 7.15 M
GAS HOLDUP = .0107

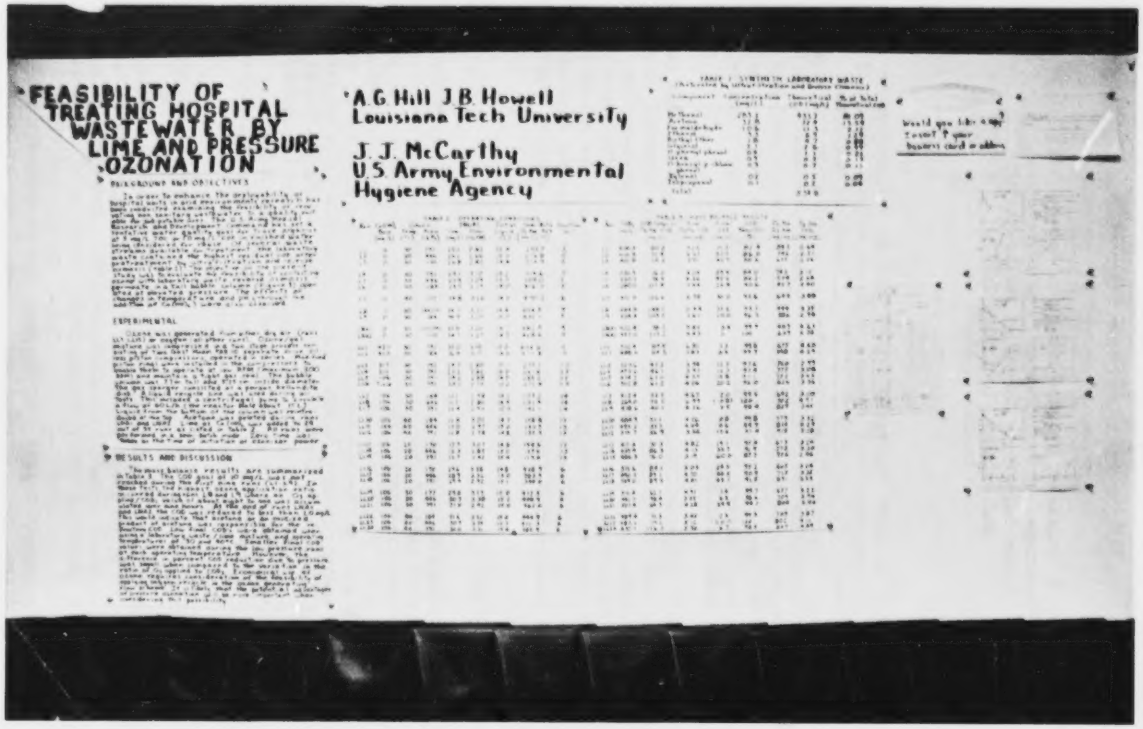
SEMI-BATCH OZONE APPLICATION RATE = 142.1 MG/(L*H)

AVERAGE EXIT GAS FLOW = 2.77 L/MIN

LIQUID RECYCLE RATE = 60 L/H

SAMPLE PORT HEIGHTS :
LEVEL2 = 2.29 M
LEVEL3 = 4.73 M
LEVEL4 = 5.95 M

Figure 4



FEASIBILITY OF TREATING HOSPITAL WASTEWATER BY LIME AND PRESSURE OZONATION

A.G. Hill J.B. Howell
Louisiana Tech University

J. J. McCarthy
U.S. Army Environmental
Hygiene Agency

BACKGROUND AND OBJECTIVES

EXPERIMENTAL

RESULTS AND DISCUSSION

TABLE 1. SYNTHETIC LABORATORY WASTE

Would you like a copy?
Leave your
business card or address

WASTEWATER REUSE - PERSISTENCE OF CHEMICAL POLLUTANTS

A. J. Englande, Jr. and Robert S. Reimers III
Department of Environmental Health Sciences
Tulane University School of Public Health
New Orleans, Louisiana 70112

INTRODUCTION

Desired quality of water for reuse varies with anticipated usage. Reuse applications consequently will require emphasis on the removal of selected physical, chemical, and biological constituents to use-specific, nonadverse levels. Treatment plant operating experience has indicated several types of contaminants, particularly chemical parameters, to be difficult and expensive to adequately remove with currently employed waste treatment technology. These persistent constituents include: nitrogen (ammonia and NO_3), phenol, total dissolved solids, trace organics, boron, and specific heavy metals (1, 2, 3).

This paper will focus on chemical pollutants (and in particular on trace metals) which tend to persist through biological and physical chemical treatment systems and factors which affect leakage (complexation, redox-potential, speciation, bio-activity, etc.). Removal efficiencies of specific unit operations for important parameters and the effect of influent variability on effluent water quality will also be assessed based on operating data derived from pilot and full scale waste treatment facilities.

METHODS AND RESULTS

Allowable contaminant content of treated effluents will, as noted, depend on intended use. Table 1 indicates pertinent water quality criteria for various use classifications. Data is analyzed using Table 1 as a basis for comparison to "flag" specific parameters of concern for reuse application. Data analyzed herein includes: 1) results of a survey comparing effluent quality from five AWT plants to potable water quality criteria; 2) long term data evaluation from pilot and full scale studies to determine the effectiveness of chemical contaminant removal by biological, tertiary (AWT) and physical-chemical processes; and 3) results of pilot and full scale studies to derive relative removal efficiencies of selective unit operations for parameters of concern.

AWT Plant Survey

Extensive chemical, physical, and biological analyses were conducted on periodic samples of effluent taken from five AWT demonstration facilities treating predominantly domestic wastewaters (8). Results were compared to current U.S. drinking water regulations. Those pilot and full scale plant sites evaluated included: Lake Tahoe, California; Blue Plains, District of Columbia; Pomona, California; Dallas, Texas; and Orange County, California. These systems were chosen primarily because of availability and because effluent quality exceeded that of secondary treatment systems. It should be noted that the Orange County facility was tested during a period of initial start-up. None of the systems were designed to produce a water of potable quality as each was part of an independent project with specific individual goals.

Spot samples taken over a six to nine month period indicated that certain chemical parameters tended to persist through the various treatment schemes and were found to exceed drinking water standards in most of the treated effluents. These parameters included: nitrogen (ammonia and nitrate), phenol, carbon chloroform extract (CCE), total dissolved solids (TDS), and specific heavy metals.

Table 2 summarizes survey data by tabulating pertinent effluent constituent concentrations and respective exceedance ratios (number of samples surpassing potable water quality/total number of spot samples evaluated). As indicated, nitrogen (ammonia and nitrate), total dissolved solids, and phenol exhibited the highest exceedance ratios of the facilities evaluated. The Blue Plains facility which employs biological nitrification-denitrification was the only system to maintain residual ammonia concentration within acceptable limits. The treatment plants studied were not designed to remove TDS and hence in most cases the TDS content increased due to chemical treatment. Phenolic compounds in levels exceeding the 1 μg/l standard were found in all effluents. Except for Orange County, CCE concentrations also exceeded the originally proposed limit of 0.7 mg/l for all cases.

As shown by Table 2, selenium appeared to be the most persistent heavy metal characterized by high exceedance ratios for effluents from all facilities except Orange County. Chromium, arsenic, and iron also were relatively stable and little removal was effected through the AWT plants evaluated. Cadmium, lead, mercury, manganese, and silver occasionally exceeded potable criteria. AWT facilities employing lime treatment reported the highest and most consistent reduction in heavy metals concentrations due to precipitation, co-precipitation, and adsorption by hydroxides and carbonates formed at high pH. Pomona, which did not employ lime treatment, experienced the highest diversity of specific heavy metal exceedance ratios. All systems, however, were characterized by high quality effluents and product water approaching potable quality.

Removal of Chemical Contaminants by Conventional Waste Treatment Technology

The spot sampling approach employed during the preceding study proved valuable in underscoring specific contaminants of potential concern including effluent variability considerations. Average long-term performance capabilities, however, were not defined. Two of the plants, Dallas and Orange County, have subsequently published data continuously collected over 18 and 6 month periods, respectively (9, 10). These data along with that from pilot and full-scale studies concerned with removal and pass through of pollutants in conventional biological, tertiary, and physical-chemical wastetreatment processes, are summarized as follows.

a) Chemical Pollutant Reduction by Biological Methods. A recent "state-of-the-art" review of trace metal removals from wastewaters by Cohen (11) indicated ranges of per cent removals obtainable by biological treatment. These results, in part, are presented in Table 3. Also shown are projected secondary treatment reductions in metals content as experimentally determined by the Sanitation District of Los Angeles County for assessing industrial effluent concentration limitations (12). Long term chemical pollutant removal efficiencies are given for the Dallas activated sludge plant for the facility previously described. Results of an EPA survey (13) of secondary treatment effectiveness for chemical contaminant removal are also included.

Results tabulated in Table 3 show a wide range of demonstrated removal efficiencies which appear largely unpredictable. Inconsistencies are partly due to the characteristics of the influent wastewaters, the complexities of the biological wastewater stabilization processes, and overall process efficiencies. The Orange County AWT plant, for example, recently observed significant improvement in effluent quality following a change in secondary treatment from trickling filter to activated sludge and with the exclusion of a high nitrogen content industrial waste from discharging into the collection system (14). The acceptability of variations in effluent quality will depend on the influent pollutant concentration, the intended use of the product water, and the dilution capacity at the point of discharge. This point is particularly illustrated by the L. A. Sanitary District results. Since significant industrial waste contributions are realized, source control will be required for chromium and zinc for compliance with ocean discharge limitations.

A comparison of nitrified Dallas effluent quality to water quality criteria for aquatic life (Table 1) indicates exceedance with respect to ammonia nitrogen, silver, cadmium, mercury, lead, and zinc. Results of an EPA survey of 163 biological treatment plants indicate aquatic life criteria exceedance with respect to ammonia, phenol, cadmium, chromium, iron, mercury, manganese, lead, and zinc. The importance of adequate dilution capacity of the receiving water is again illustrated. Since the Dallas municipal plant is servicing approximately 12% industrial flow by volume and the plants surveyed by EPA an unknown industrial contribution, any further industrial increase would warrant added concern.

b) Chemical Pollutant Removal by AWT Methods. The treatment system which shows most promise in effectively achieving an effluent of consistently high quality is a combination of biological and physical-chemical processes. Table 4 illustrates overall tertiary, physical-chemical treatment removals of chemical contaminants from the Dallas and Orange County studies. The treatment sequence employed at Dallas following nitrifying activated sludge includes high lime coagulation (pH=11) and clarification, recarbonation, mixed media filtration, carbon adsorption, and chlorination; whereas, that at Orange County following trickling filtration consists of high lime coagulation (pH>11) and clarification, ammonia stripping, two-stage recarbonation, mixed media filtration, activated carbon adsorption, and chlorination. Both plants treat wastewaters of low industrial content as demonstrated by the low incoming concentrations of metals.

As indicated in Table 4, treated effluents were generally of excellent quality especially with respect to potable criteria. Some difficulties, however, are experienced in meeting stringent requirements for aquatic life, particularly if only minimum dilution is provided by the receiving water. Aquatic life criteria were exceeded at Dallas for ammonia, silver, cadmium, mercury, lead, and zinc. Orange County experienced exceedance for aquatic quality (ammonia, phenol, silver, cadmium, mercury, and zinc), groundwater injection (ammonia, phenol and boron), drinking water (ammonia and mercury), and irrigation (boron).

Relatively poor overall removals (<50%) were noted at Dallas for TOC (44.2%), nitrite and nitrate nitrogen (0%), TDS (Increase), silver (7.7%), boron (10%), barium (Increase), chromium (25.9%), copper (Increase), mercury (Increase), lead (Increase) and zinc (0%). Orange County reported poor reductions for arsenic (12.1%), boron (16%), mercury (22.8%), and selenium (Increase). Comparison of removal percentages for metals, however, may be misleading since with low incoming concentrations, the numerical value of the residual metal content becomes increasingly sensitive to solubility product relationships, concentration gradients, and the efficiency of suspended solids removal. It should also be noted that in many cases residual concentrations approached limits of detection.

As will be subsequently discussed in more detail, the degree of metals reduction is plant specific and dependent upon the specific metal, the presence of complexing constituents, oxidation-reduction potential, pH, carbon activity, coagulant employed, ionic makeup of carrier water, and other operating and environmental factors.

c) Chemical Pollutant Removal by Physical-Chemical Treatment. Due to a lack of available data, only metals removal efficiency by physical-chemical treatment will be evaluated. Physical-chemical systems as demonstrated by Cohen (11) are generally more efficient than biological treatment in reducing variability and concentrations of metals in effluents. A 4 gpm pilot plant consisting of chemical coagulation and clarification, dual media filtration, and activated carbon was evaluated by spiking raw municipal waste water with elevated metals concentrations. The effect of various coagulants including lime, alum, and ferric chloride on metals reduction was investigated. Results are summarized in Table 5.

Physical-chemical treatment is shown to be a most effective series of processes for reduction of metals from wastewaters. Coagulation with one or another of the coagulants followed by activated carbon obtained removal efficiencies of greater than 95% for most metals.

Degree of removal was found to be dependent on the type of coagulant employed. Manganese, for example, was poorly removed by iron salts or alum, but readily precipitated by lime. The influence of influent concentrations was illustrated by selenium data. Removals decreased significantly as the initial Se level was reduced from 0.5 to 0.06 mg/l. It was concluded that while high removal efficiencies were reported, the stringent requirements for aquatic life would not be met for many metals particularly at high influent content. Alternative control measures, for example source control, would be required.

Removal of Chemical Pollutants by Selected Unit Operations

To better interpret overall chemical contaminant removal results from AWT and physical-chemical systems, it is necessary to evaluate reduction efficiencies of specific unit operations. The effects of precipitation, filtration, and carbon adsorption will be evaluated by summarizing data from Dallas, Orange County, and that of Hannah and Cohen (15). Results from the latter study were derived from the same system as discussed by Cohen and described in the preceding section. Two carbon contactors were used: the "old carbon" having been in operation for about one year and in need of regeneration; the other was designed as "new carbon" characterized by minimal bioactivity.

Tables 6, 7 and 8 illustrate per cent decreases with corresponding residual parameter concentrations for precipitation, filtration, and carbon adsorption processes. Filtration is used as a polishing step to precipitation - clarification and is not effective in soluble reductions. Both processes were evaluated as one at the Dallas Facility. The effectiveness of these unit operations will be discussed in detail in the subsequent Discussion of Results Section.

DISCUSSION OF RESULTS

Results will be interpreted on an individual parameter basis. Reference should be made to Table 9 which summarizes pertinent chemical removal efficiencies for selected unit operations.

Total Dissolved Solids

By nature of the treatment process, accumulation of mineral content is inherent in water reuse. Table 6 indicates that treatment by the Dallas facility resulted in a net increase in TDS due primarily to lime addition during chemical precipitation. Influent waters sufficiently high in TDS will consequently require suitable blending water sources prior to reuse. Recent experience at Orange County has demonstrated reverse osmosis to be effective and feasible in removing the dissolved salt content of a portion of the effluent with subsequent blending with the remaining portion following activated carbon treatment (10).

Nitrogen Removal

Even though high overall ammonia removal efficiencies were experienced at Dallas (67%) by the nitrifying activated sludge process and at Orange County (82%) by series operated ammonia stripping towers, residual concentrations remain above the criteria limits set for freshwater aquatic life and groundwater injection. Breakpoint chlorination practiced at Orange County resulted in reduction of ammonia nitrogen to the required 1 mg/l level for injection. Because of high operating cost and the formation of several chlorinated organics not effectively removed by activated carbon, the viability of this alternative is questionable. It should be noted that Orange County performance reflects operation with high influent NH_3-N concentrations of 43 mg/l.

The Blue Plains Treatment System 2 (low lime clarification, dispersed growth nitrification, denitrification, carbon adsorption, and mixed media filtration) maintained residual ammonia concentrations within acceptable limits. Based on this data, biological nitrification-denitrification appears to offer a promising alternative for nitrogen removal.

Phenol

Phenolic compounds in levels exceeding the 1 μg/l standard were found in all effluents studied. Biological-AWT methods as currently employed appear to be inadequate in removing phenol to the 1 μg/l requirement.

Trace Organics

Residual COD and TOC concentrations are often used as indicators of trace organic effluent content. As shown in Tables 6 and 8 high removals (>50%) of COD were experienced at both Dallas and Orange County by precipitation and carbon adsorption methods. While TOC removal efficiency varied through each facility, residual concentrations of 6.7 mg/l TOC were identical. The CCE content is also sometimes employed as a measure of trace organics. This too does not represent any indication of individual organics which may or may not be of toxicological significance. Due to the inaccuracy inherent in the CCE procedure and the questionable significance of the values obtained, this parameter was dropped as a potable water quality parameter.

Besides clarification and activated-carbon adsorption, recent work by McCarty et al. (10) at the Orange County facility demonstrates that air stripping for ammonia removal is also highly effective in removing a wide range of highly volatile, low molecular weight refractory organics. These organics are relatively nonpolar which are not easily removed by activated carbon or reverse osmosis. Breakpoint chlorination, on the other hand, results in the formation of chlorinated organics, the health significance of which requires further investigation.

Specific Heavy Metals

Arsenic. Efficiency of physical-chemical methods for arsenic removal depends greatly on initial concentration. Precipitation with ferric chloride achieved approximately 90% removal, and activated carbon increased reductions to about 97%. Similar treatment with lime resulted in an 84% overall reduction of arsenic at an initial concentration of 5 mg/l (Table 5). Precipitation efficiency was particularly low (24%) at the Orange County AWT plant, however, and carbon adsorption removal was negligible at both Orange County and Dallas. An effluent residual of approximately 3 μg/l was reported by both facilities. Reduced removals are due in part to much lower concentration gradients. These low μg/l residual values are near the detection limit.

Maruyama et al. (16) reported arsenic to form slightly soluble compounds with a number of metals, including iron. Insoluble arsenic trisulfide was found to precipitate by reaction with hydrogen sulfide in acid solution, but readily dissolve in basic solutions. Iron precipitation was recommended for arsenic reduction.

Barium. Effectiveness of physical-chemical treatment processes for barium removal increases with greater influent barium and sulfate levels. Barium forms slightly soluble barium carbonate and barium sulfate complexes. The hydroxide is relatively soluble and not thought involved in the removal process. Low lime, iron and alum precipitation was found efficient in removing barium by 99.5, 95 and 92% respectively. High lime addition resulted in lower reductions (81% at an initial concentration of 5 mg/l) due to the formation of relatively soluble barium hydroxide.

Removal of barium by AWT processes was not as promising as indicated by the above results. Reductions varied from less than 10% at Dallas to 56% at Orange County by high lime precipitation. Activated carbon treatment resulted in negligible removals (<10%) as predicted by Sigworth and Smith (17) due to its highly soluble nature. Enhanced precipitation reductions at Orange County are attributed to the relatively high influent sulfate concentration of 334 mg/l compared to 98 mg/l at Dallas.

Boron. The highly soluble and complexing properties of boron characterize it as a persistent element through both biological and chemical processes. Data indicates negligible removals through both the Dallas and Orange County AWT facilities. Residual effluent concentrations were 290 μg/l (<10% removed) and 840 μg/l (16% reduction) respectively.

Cadmium. Physical-chemical processes employing lime and ferric chloride as coagulants experienced high removals of cadmium (99.6% and 98.6% respectively). When alum was employed the removal efficiency decreased to 55%. Best removals were observed for the high lime system with aged carbon. This result was not unexpected, since cadmium forms an insoluble hydroxide at high pH and is precipitated by sulfide over a broad pH range. Cadmium may form soluble complexes with ammonia and cyanide which can reduce precipitation removal efficiency. High lime precipitation followed by carbon adsorption at the AWT facilities resulted in what appears to be a minimum cadmium solubility for these processes at approximately 2 μg/l.

Chromium. Removal of chromium will depend primarily on its oxidation state and initial concentration. Trivalent chromium behaves much differently than the hexavalent form and is much less soluble. Hydroxide ions readily react with Cr^{+3} to precipitate hydrous oxide which is amphoteric dissolving in excess base. As expected, good removals of chromium (>50%) were observed by precipitation in physical-chemical and AWT systems. Filtration effectively removed trivalent chromium, but the soluble Cr^{+6} was relatively unaffected. Chromium removal efficiencies decreased with lower influent concentrations as illustrated by comparing AWT and physical-chemical results (except for alum coagulation). Carbon adsorption was effective in reducing Cr^{+6} content by reduction to the less soluble trivalent state. Reduction is probably due to chemical and biological activity within the carbon column. Data appears to indicate that below a residual of 20 μg/l, the sorption of chromium becomes negligible.

Copper. Good removals were experienced in removing copper from waste waters by physical-chemical processes. Chemical precipitation using iron and low lime resulted in 96 and 93% reductions respectively with influent concentrations of 5 mg/l. High lime removals were somewhat erratic averaging about 90%. Alum was the least effective of the coagulants employed, resulting in only a 70% reduction at an influent concentration of 0.7 mg/l. Activated carbon appeared more efficient with alum addition increasing removals to 98%. New carbon exhibited better removal capacity than aged carbon.

The AWT facilities while characterized by lower removal efficiencies also produced lower residual metals content which again indicates the significance of influent concentration and minimum solubility levels. High lime precipitation indicated copper to have a limiting solubility of approximately 70 μg/l at a pH of 11.5. Data also indicates some affinity of copper for carbon in AWT systems (>35% removal) contrary to other findings (17). Enhanced sorption could be due to the formation of complexes which may be sorbed to the carbon or biological growth within the column.

Iron. High levels of residual iron in treated effluents are primarily the result of $FeCl_3$ or $FeSO_4$ addition to enhance sedimentation. Chemical precipitation can be effective in reducing iron content depending on pH, oxidation-reduction potential, and initial concentration. Iron can easily be precipitated as ferric hydroxide at high pH under aerobic conditions. High lime coagulation at the AWT facilities resulted in residual iron levels of 120 μg/l and 40 μg/l for Dallas and Orange County respectively. Bioactivity within carbon columns can reduce insoluble Fe^{+3} to soluble Fe^{+2} causing release. Added complexity results, however, since iron can react within the column to form insoluble iron sulfides. More than 68% of iron was removed by carbon adsorption at Orange County; whereas, less than half that efficiency was observed at the Dallas facility.

Lead. Physical-chemical treatment gave excellent removals of lead yielding approximately 99% reduction at an initial level of 5.0 mg/l. At reduced concentrations, however, variable efficiencies were observed. Dallas and Orange County reported overall per cent removals of less than 10% and 72% respectively. Fluctuations may be due to organic complexation, chamical speciation, and solubility limitations. Lead also forms insoluble salts with sulfate, carbonate, and sulfides which may account for the enhanced Orange County removals by precipitation (81%) since this waste water was characterized by a higher initial sulfate content. Negligible removals were observed by activated carbon treatment.

Manganese. The effect of coagulant selection on manganese removal was demonstrated by Cohen (11) where high removals with lime (>90%), but poorer reductions with alum and $FeCl_3$ (<3%) were observed during precipitation. Inorganic manganese salts are soluble in acid and neutral solution but form insoluble manganous hydroxide precipitates with increasing pH. Under alkaline and aerobic conditions Mn^{+2} will be oxidized to insoluble manganese dioxide. Precipitation results are therefore as expected. A residual concentration of about 5 μg/l appears to represent the lowest concentration effectively removed by adsorption. The reducing conditions within the carbon column may cause reduction to the soluble divalent manganese ion with subsequent release.

Mercury. Physical-chemical removal of mercury is influenced by pH, initial mercury content, and degree of complexation. Precipitation in physical-chemical plants was found very effective (>90% removal) when alum and ferric chloride were used as coagulants. When lime was employed, however, removal decreased to 45% at an initial mercury level of 500 μg/l. AWT plants indicated low and inconsistent reductions through each unit operation at the low residual concentrations present (<5 μg/l). At pH values above 8.5 the formation of relatively soluble mercuric oxide may result in poorer removals. Filtration can be used to enhance recovery since most mercury precipitates are not easily removed by clarification. Sorption efficiency of activated carbon drops markedly as the influent level decreases to below 5 μg/l. Due to its nonionic properties and its affinity for sulfhydryl groups carbon adsorption may be effective in mercury removal. Bioactivity resulting in methylation or demethylation will also reduce adsorption capacity. As indicated by Table 4, mercury was not removed to any significant degree at either the Dallas or Orange County facilities.

Selenium. None of the coagulants tested were highly effective in removing selenium by coagulation-filtration. Activated carbon performance was unpredictable. Good removals (except for alum) were attained with sorption until influent carbon levels of selenium decreased to less than 20 μg/l. Higher effluent concentrations were observed when alum was employed due to the formation of selenate complexes or perhaps competitive sorption of the chemically similar sulfur compounds present.

Silver. Physical-chemical treatment of elevated silver concentrations (0.5 mg/l) produced very high removals of silver (>90%) independent of the coagulant used. At the lower influent levels experienced during AWT systems operation removals were variable. Residual concentrations, however, were less than 4 μg/l. Carbon adsorption proved fairly effective in all systems studied (except for Orange County where low residuals of 1.5 μg/l existed) due to the relatively high affinity of silver for reduced sulfhydryl groups. Data indicates that for the processes and conditions evaluated herein an apparent limiting silver solubility of about 2 μg/l exists.

Zinc. The amphoteric properties of zinc affect precipitation results. Using alum as a coagulant resulted in a pH reduction (6 to 7) with only 25% of the zinc being precipitated. $FeCl_3$ application resulted in a 63% removal, whereas with low lime addition better removals were observed in the physical-chemical system (94%) but more variability in effluent zinc content was noted due to the formation of soluble zinc complexes (12). At lower influent concentrations experienced by the AWT facilities (63 μg/l and 300 μg/l for Dallas and Orange County respectively) overall plant removals ranged from 0 to 59%. None of the unit operations were significantly effective in reducing zinc content except at the Orange County plant where carbon adsorption provided a 61% reduction. This may be due to zinc sulfide precipitation in the carbon columns.

CONCLUSIONS

Data from biological, tertiary, and physical-chemical treatment processes illustrate the necessity of a systems approach in design of wastewater renovation facilities. To attain maximum effectiveness and reliability, the treatment system must consist of several unit operations in series with overlapping functions that rely on different and unrelated removal mechanisms. Biological treatment, for example, while not particularly effective in reducing pollutants to trace levels, can decrease the amount of pollutant to be handled by subsequent tertiary treatment; and very importantly, minimize influent variability by providing buffer capacity. The coagulant selected in the treatment sequence can greatly affect removal efficiencies by coagulation and subsequent treatment processes. High lime was found to be most effective for removal of the various metals studied. The recommended coagulant, however, should be metal and system specific.

Chemical parameters found to consistently exceed water quality criteria for intended use included: total dissolved solids, nitrogen (ammonia and nitrate), phenol, trace organics, and specific heavy metals. Total dissolved solids were not removed to any appreciable extent by the treatment sequences evaluated except for the reverse osmosis facility operated at Orange County. Ammonia nitrogen was removed to a level equal to or less than 1 mg/l only when breakpoint chlorination polished stripping tower effluent or when biological nitrification-denitrification was employed. Concern, however, exists over health effects of chlorinated organics generated by the breakpoint chlorination process. Significant organic removal was effected by chemical precipitation, carbon adsorption, and ammonia stripping. The air stripping process was found highly effective in removing a wide range of highly volatile, low molecular weight refractory organics not easily removed by activated carbon or reverse osmosis. None of the treatment sequences evaluated appeared capable of reducing phenol concentrations to a 1 μg/l limit. The preceding results underscore the importance of suitable blending water sources prior to reuse.

In general AWT and physical-chemical facilities produced effluents of excellent metals quality, especially with respect to potable water criteria. Metals found to persist through the treatment schemes evaluated included boron, mercury, selenium, and zinc. Removal of many metals is inconsistent due to metal concentration, nature of carrier water, etc. Residual concentrations of boron and zinc exceeded recommended criteria for irrigation and aquatic life respectively. Data appeared to indicate minimum metals residual concentrations obtainable by the processes studied. Residuals of arsenic, cadmium, chromium, mercury, selenium, and silver were found to be 3 μg/l, 2 μg/l, 20 μg/l, 5 μg/l, 2-20 μg/l, and 2 μg/l respectively. It should be noted that most of these values are near their respective limits of detection. These values exceed potable water standards for selenium and mercury; ground water injection recommendations for selenium; and aquatic life criteria for cadmium, mercury, selenium, and silver. The importance of adequate dilution capacity of the receiving water is therefore self evident.

A significant decrease in removal efficiency was observed with reduced influent metals concentration. It should be noted, however, that comparison of removal percentages may be misleading since with low incoming concentrations the numerical value of the residual metals content becomes increasingly sensitive to solubility product relationships, concentration gradients, and the efficiency of suspended solids removal. High initial levels of metals could result in unacceptable effluent concentrations of specific metals, particularly in plants experiencing significant industrial input. Alternative methods of reduction such as source control will be required for such cases.

REFERENCES

1. ENGLANDE, A. J.; SMITH, J. K.; & ENGLISH, J. N. Potable Water Quality of Advanced Wastewater Treatment Plant Effluents, paper presented at the IAWPR Workshop in Advanced Treatment and Reclamation of Wastewater, South Africa (1977).

2. IDELOVITCH, E. Wastewater Reuse by Biological-chemical Treatment and Groundwater Recharge, paper presented at the Water Pollution Control Federation Meeting, Philadelphia, Pennsylvania, October 2-7, 1977.

3. HAHINGH, W. H. J. Reclaimed Water: a Health Hazard? Water South Africa 3:104 (1977).

4. National Interim Primary Drinking Water Regulations. Federal Register, 40(248): 59565 (December 24, 1975).

5. Ecological Research Series. Water Quality Criteria 1972. A report of the Committee on Water Quality Criteria, Washington, D.C., 1972.

6. U. S. Environmental Protection Agency. Quality Criteria for Water, EPA-440/9-76-023, Washington, D.C., 1976.

7. ARGO, DAVID G. The Cost of Water Reclamation by Advanced Wastewater Treatment, paper presented at the Water Pollution Control Federation 51st Annual Conference, Anaheim, California, October 1-6, 1978.

8. SMITH, J. K.: ENGLANDE, A. J., JR.; MCKOWN, M. M.; & LYNCH, S. C. Characterization of Reuseable Municipal Wastewater Effluents and Concentration of Organic Constituents. U.S.E.P.A. (EPA-600/2-78-016), Cincinnati, Ohio (February, 1978).

9. Municipal Wastewater Reuse News. American Water Works Research Foundation 6:6, Denver, Colorado (March, 1978).

10. MCCARTY, P. L.; REINHARD, M.; DOLU, C.; NGUYEN, H.; & ARGO, D. G. Water Factory 21: Inorganic, Organic, and Biological Quality of Reclaimed Wastewater and Plant Performance. Department of Civil Engineering, Standford University, Technical Report No. 226, January, 1978.

11. COHEN, J. M. Trace Metal Removal by Wastewater Treatment. USEPA Technology Transfer (January, 1977)

12. KREMER, J. G. & THOMAS, J. D. The Industrial Waste Program of the Los Angeles County Sanitation Districts. Water Pollution Control Federation Highlights, 14 (August, 1977).

13. U. S. Environmental Protection Agency. State and Local Pretreatment Programs. EPA 430 19-76-0176, Washington, D.C., January 1977.

14. ARGO, DAVID G. Orange County Water District personal communication January, 1979.

15. HANNAH, S. A. & COHEN, J. M. Removal of Uncommon Trace Metals by Physical and Chemical Treatment Processes. Journal Water Pollution Control 49:2297 (November, 1977).

16. MARUYAMA, T.; HANNAH, S. A.: & COHEN, J. M. Metal Removal by Physican and Chemical Treatment Processes. Journal of WPCF 47:5:962-975 (May 1975).

17. SIGWORTH, E. A. & SMITH, S. B. Adsorption of Inorganic Compounds by Activated Carbon. Taste and Odor Journal 38(3rd quarter):1-7 (1972).

TABLE 1

Water Quality Criteria for Selected Chemical Parameters

Parameter	Drinking Water Permissible Levels (4)	Aquatic Water (5, 6) Minimum Risk to Hazard Level	Groundwater Injection (7)	Irrigation (5)
Ammonia Nitrogen (N)	0.5 mg/l*	0.01 to 0.4 mg/l	1.0 mg/l	-
Total Nitrogen (N)	-	-	10 mg/l	-
Nitrates (as N)	10 mg/l	-	-	-
Phenol	-	1 μg/l	1 μg/l	-
Total Dissolved Solids (TDS)	500 mg/l	-	-	-
COD	-	-	30 mg/l	-
CCE	70 μg/l**	-	-	-
Ag	50 μg/l	1 to 5.0 μg/l	50 μg/l	-
As	50 μg/l	-	50 μg/l	100 μg/l
B	-	-	0.5 mg/l	750 μg/l
Ba	1000 μg/l	-	1000 μg/l	-
Cd	10 μg/l	0.4 to 1.2 μg/l (soft to hard waters) 5 μg/l marine	10 μg/l	10 μg/l
Cr^{+6}	50 μg/l	100 μg/l	50 μg/l	100 μg/l
Cu	1000 μg/l	0.1 X 96 hour LC50	1000 μg/l	200 μg/l
Fe	300 μg/l	1.0 mg/l	300 μg/l	5 mg/l
Hg	2 μg/l	0.05 μg/l to 0.10 μg/l (fresh to marine water)	5 μg/l	-
Mn	50 μg/l	100 μg/l (marine)	50 μg/l	200 μg/l
Pb	50 μg/l	30 μg/l	50 μg/l	5 mg/l
Se	10 μg/l	5 to 10 μg/l	10 μg/l	20 μg/l
Zn	5000 μg/l	20 to 100 μg/l	-	2.0 mg/l

*Permissible Criteria for Public Water Supplies from Water Quality Criteria April 1, 1968, Federal Water Pollution Control Adm., Washington, D.C.

**Proposed, but omitted in final standards

TABLE 2

Potable Water Quality Exceedance Ratios (E.R.) for AWT Plant Spot Survey

LAKE TAHOE			BLUE PLAINS						ORANGE COUNTY		
			System 1			System 2					
Parameter	Median	E.R.	Parameter	Median	E.R.	Parameter	Median	E.R.	Parameter	Median	E.R.
NH_3-N	15 mg/l	13/13	TDS	524 mg/l	3/3	Se	10 μg/l	1/6	NH_3-N	14.2 mg/l	6/6
Se	16 μg/l	13/13	CCE	1300 μg/l	2/2	NO_3-N	0.5 mg/l	1/6	TDS	937 mg/l	6/6
Phenol	5 μg/l	7/12	NH_3-N	0.65 mg/l	2/3	Phenol	<5 μg/l	1/6	Cr	57 μg/l	5/6
CCE	650 μg/l	3/8	Phenol	<5 μg/l	1/2				NO_3-N	11.3 mg/l	5/6
Fe	140 μg/l	3/13	Cr	19 μg/l	1/3				Phenol	<5 μg/l	1/6
As	<10 μg/l	2/13	Pb	19 μg/l	1/3						
Cd	<1 μg/l	1/13									
TDS	414 mg/l	1/13									

DALLAS			POMONA								
			System 1			System 2			System 3		
Parameter	Median	E.R.	Parameter	Median	E.R.	Parameter	Median	E.R.	Parameter	Median	E.R.
Phenol	7.2 μg/l	6/8	TDS	513 mg/l	7/9	NH_3-N	7.0 mg/l	8/9	NH_3-N	4.7 mg/l	9/9
Se	35 μg/l	5/8	NH_3-N	2.0 mg/l	5/9	TDS	538 mg/l	6/9	TDS	582 mg/l	8/9
TDS	478 mg/l	3/8	Se	<10 μg/l	5/9	Phenol	<5 μg/l	3/9	Cr	55 μg/l	5/9
As	21 μg/l	2/7	CCE	650 μg/l	4/8	NO_3-N	2.7 mg/l	3/9	NO_3-N	8.0 mg/l	4/9
NH_3-N	0.08 mg/l	2/8	Phenol	<5 μg/l	3/9	As	<10 μg/l	2/9	Phenol	5 μg/l	3/9
CCE	200 μg/l	1/7	Cr	30 μg/l	2/9	CCE	340 μg/l	1/8	CCE	400 μg/l	3/9
Cr	4 μg/l	1/8	NO_3-N	4.5 mg/l	2/9	Fe	40 μg/l	1/9	Ag	41 μg/l	2/9
Fe	93 μg/l	1/8	Fe	7.9 μg/l	2/9						
			Hg	0.31 μg/l	2/9						
			Mn	0.0 μg/l	1/9						

Lake Tahoe - Trickling filter (TF), nitrifying activated sludge (NAS), high lime coagulation (HLC) and clarification (C), filtration (F), carbon adsorption (CA).

Blue Plains: System 1 - Low lime clarification (LLC), breakpoint chlorination (BC), CA, dual media filtration.
System 2 - LLC, dispersed growth nitrification, denitrification, CA, mixed media filtration.

Orange County - TF, HLC and C, ammonia stripping, F, activated carbon (AC).

Dallas - Activated sludge (AS), HLC and C, F, CA.

Pomona: System 1 - AS, AC. System 2 - AS, AC, chlorination, AC. System 3 - AS, AC, ozonation, AC.

TABLE 3

Removal of Selected Contaminants by Biological Treatment

	Ranges of Removals as Reported by Cohen (11)	LA Sanitary District Projected Removals (12)		Activated Sludge Removal Dallas (9)		EPA Study Removal (13)	
		Residual Metal		Residual Parameter		Residual Parameter	
	Per Cent	Per Cent	(μg/1)	Per Cent	Concentration	Per Cent	Concentration
TOC	-	-	-	70.7	12 mg/1	60.0	25 mg/1
COD	-	-	-	80.7	50 mg/1	73.0	110 mg/1
NH_3-N	-	-	-	67.3	5 mg/1	42.0	14 mg/1
TKN	-	-	-	61.3	9 mg/1	34.0	18 mg/1
NO_3 & NO_3-N	-	-	-	Inc.**	5.1 mg/1	-	-
Phenol	-	-	-	-	-	45.0	175 μg/1
Ag	-	69	5.3	Inc.**	2.6 μg/1	-	-
As	-	48	5.2	18.7	17.0 μg/1	-	-
B	-	-	-	Inc.**	300 μg/1	-	-
Ba	-	-	-	33.3	120 μg/1	-	-
Cd	20-45	73	5.7	58.3	5 μg/1	18	30 μg/1
Cr	40-80	77	240*	64.9	27 μg/1	42	218 μg/1
Cu	0-70	76	-	79.4	29 μg/1	56	113 μg/1
Fe	-	-	-	9.2	590 μg/1	57	1827 μg/1
Hg	20-75	84	0.19	44.8	0.16 μg/1	35	3.5 μg/1
Mn	-	-	-	42.3	41 μg/1	35	140 μg/1
Ni	-	-	-	-	-	21	182 μg/1
Pb	50-90	80	58	52.8	34 μg/1	38	92 μg/1
Se	-	-	-	34.1	2.9 μg/1	-	-
Zn	35-80	77	497*	44.2	63 μg/1	52	277 μg/1

*Source controls needed to meet ocean outfall criteria

**Inc. - Increase

TABLE 4

Removal of Selected Parameters by Tertiary Physical-Chemical Treatment

PARAMETER	DALLAS (9)			ORANGE COUNTY (10)		
	Initial Concentration	Removal Per Cent	Residual Concentration	Initial Concentration	Removal Per Cent	Residual Concentration
TOC	12 mg/l	44.2	6.7 mg/l	-	-	6.7 mg/l
COD	50 mg/l	92.0	4.0 mg/l	142 mg/l	87.3	18 mg/l
NH_3-N	5 mg/l	28.0	3.6 mg/l	45 mg/l	93.0	3.1 mg/l
TKN	9 mg/l	50.0	4.5 mg/l	53 mg/l	91.0	4.8 mg/l
NO_2 & NO_3-N	5.1 mg/l	0	5.1 mg/l	-	-	-
TDS	479 mg/l	Inc.*	608 mg/l	1020 mg/l	-	-
Phenol	-	-	-	-	-	3.9 μg/l
Ag	2.6 μg/l	7.7	2.4 μg/l	5.5 μg/l	73.0	1.5 μg/l
As	17.0 μg/l	82.3	3.0 μg/l	3.3 μg/l	27.0	2.4 μg/l
B	300 μg/l	10.0	270 μg/l	1000 μg/l	16.0	840 μg/l
Ba	120 μg/l	Inc.*	140 μg/l	81 μg/l	62.0	31.0 μg/l
Cd	5 μg/l	60.0	2 μg/l	29 μg/l	94.0	1.7 μg/l
Cr	27 μg/l	25.9	20 μg/l	154 μg/l	83.0	26.0 μg/l
Cu	29 μg/l	Inc.*	46 μg/l	266 μg/l	88.0	32.0 μg/l
Fe	590 μg/l	83.0	100 μg/l	325 μg/l	80.0	66.0 μg/l
Hg	0.16 μg/l	Inc.*	0.51 μg/l	9 μg/l	26.0	6.7 μg/l
Mn	41 μg/l	73.1	11 μg/l	35 μg/l	86.0	4.9 μg/l
Pb	34 μg/l	Inc.*	35 μg/l	19 μg/l	72.0	5.3 μg/l
Se	2.9 μg/l	69.0	0.9 μg/l	1.8 μg/l	Inc.*	1.9 μg/l
Zn	63 μg/l	0	63 μg/l	412 μg/l	57.0	162 μg/l

*Inc. - Increase

TABLE 5

Removal of Trace Metals by Physical-Chemical Processes (11)

	LIME PRECIPITATION-ACTIVATED CARBON			ALUM-ACTIVATED CARBON			FERRIC CHLORIDE-ACTIVATED CARBON		
METAL	Initial Concentration (mg/1)	Removal Per Cent	Residual Metal (μg/1)	Initial Concentration (mg/1)	Removal Per Cent	Residual Metal (μg/1)	Initial Concentration (mg/1)	Removal Per Cent	Residual Metal (μg/1)
Ag	0.5	98	10	0.6	99.2	5	0.5	99.1	5
As	5.0	84	800	-	-	-	5.0	97.1	145
Ba	5.0	81	950	0.5	92	40	5.0	95.6	220
Cd	5.0	99.6	20	0.7	55.5	312	5.0	98.6	70
Cr	5.0	98.2	90	0.7 Cr^{+3}	99.1	6	5.0	99.3	35
				0.7 Cr^{+6}	97.4	18			
Cu	5.0	90	500	0.7	98.3	12	5.0	96	200
Hg	0.5	92	40	0.06	98.3	1	0.05	99	1
Mn	5.0	98.2	90	0.7	33	469	5.0	17	4150
Pb	5.0	99.4	30	0.6	96.6	20	5.0	99.1	45
Se	0.5	95	25	0.5	56	220	0.1	80	20
Se	0.06	67	20	-	-	-	0.05	75	13
Zn	5.0	76	1200	2.5	28	1800	5.0	94	300

TABLE 6

Removal of Selected Parameters by Precipitation

REMOVAL Per Cent	DALLAS* (9) High Lime	ORANGE COUNTY (10) High Lime	HANNAH et al. (15)		
			High Lime	Alum	FeCl
< 10	TOC (10.9 mg/1) TDS (622 mg/1)** NO_2 & NO_3 (4.9 mg/1) Pb (34 μg/1)Ψ Ag (4.0 μg/1)** Zn (68 μg/1)** B (290 μg/1) Ba (140 μg/1)** Cu (91 μg/1)** Fe (120 μg/1)	Se (1.8 μg/1)			
10-29	NH_3-N (4.0 mg/1) Hg (0.139 μg/1)	NH_3-N (37 mg/1) TKN (41 mg/1) As (2.5 μg/1) B (840 μg/1)		Zn (1900 μg/1)	
30-49	TKN (5.9 mg/1) Cd (3 μg/1)	Zn (239 μg/1)	Hg (275 μg/1) Se (320 μg/1)† Se (32 μg/1)	Cd (400 μg/1)	
50-69	COD (18 mg/1) Cr (12 μg/1)	COD (52 mg/1) Ba (36 μg/1)		Se (235 μg/1)† Cr^{+6} (273 μg/1)	Se (34 μg/1)† Se (16 μg/1)
70-84	As (4.1 μg/1) Mn (11 μg/1) Se (0.5 μg/1)	Hg (2.6 μg/1) Cr (37 μg/1) Cu (73 μg/1) Pb (3.6 μg/1) Ag (0.8 μg/1)		Cu (210 μg/1)	
86-100		Cd (2.4 μg/1) Fe (40 μg/1) Mn (4.4 μg/1)	Ag (21 μg/1)	Ag (30 μg/1) Hg (7 μg/1) Ba (65 μg/1) Pb (54 μg/1) Cr^{+3} (49 μg/1)	Ag (30 μg/1 Hg (4 μg/1)

*includes effect of filtration

**increase

Ψvalues in parenthesis indicate residual concentration

†higher initial concentration

TABLE 7

Removal of Selected Parameters by Filtration

REMOVAL per cent	ORANGE COUNTY†† (10) High Lime	HANNAH *et al.* (15) High Lime	Alum	FeCl$_3$
<10	Cr (41 µg/l)** Zn (412 µg/l)** Fe (207 µg/l)** Pb (4.7 µg/l)** Mn (6.2 µg/l)** Ag (1.3 µg/l)** Se (1.8 µg/l) Hg (3.6 µg/l)*†	Se (325 µg/l)† Se (37 µg/l)	Se (260 µg/l)** Zn (1900 µg/l) Cu (231 µg/l)** Cd (385 µg/l) Ba (105 µg/l)** Cr^{+6} (252 µg/l)	
10-29	COD (45 mg/l) Ba (31 µg/l) Cd (1.8 µg/l) As (1.8 µg/l)			Se (25 µg/l)†
30-49	Cu (49 µg/l)	Ag (12.5 µg/l) Hg (150 µg/l)	Ag (18.6 µg/l) Hg (3.6 µg/l)	Se (10 µg/l)
50-69			Pb (27 µg/l) Cr^{+3} (17 µg/l)	
70-84	NH$_3$-N (6.4 mg/l)			Ag (9 µg/l) Hg (1 µg/l)
86-100				

† higher initial concentration

** increase

†† includes any removal or addition due to ammonia stripping and recarbonation

TABLE 8

Removal of Selected Parameters by Carbon Adsorption

REMOVAL	DALLAS (9)	ORANGE COUNTY (10)	HANNAH *et al.* (15)					
			High Lime		Alum		$FeCl_3$	
per cent	High Lime	High Lime	Old Carbon	New Carbon	Old Carbon	New Carbon	Old Carbon	New Carbon
10	TDS(608 mg/l) NH_3-N(3.6 mg/l) NO_2 & NO_3(5.1 mg/l)** Cr(20 μg/l)** B(270 μg/l) Ba(140 μg/l) Hg(0.51 μg/l)** Mn(11 μg/l) Pb(35 μg/l)** Se(0.9 μg/l)** Zn(63 μg/l)	As(2.4 μg/l)** Ba(31 μg/l) Hg(6.7 μg/l)** Se(1.9 μg/l)** Cd(1.7 μg/l) Ag(1.5 μg/l)** Pb(5.3 μg/l)**	Ag(13.5 μg/l)			Zn(1800 μg/l)	Hg(1 μg/l) Se(12 μg/l)	Hg(1 μg/l) Se(13 μg/l)
10-29	TOC(6.7 mg/l) TKN(4.5 mg/l) As(3.0 μg/l) Fe(100 μg/l)	Mn(4.9 μg/l)		Ag(10.0 μg/l)	Zn(1600 μg/l) Pb(21 μg/l)	Se(220 μg/l) Cd(312 μg/l) Pb(20 μg/l)	Se(22 μg/l)†	Se(20 μg/l)†
30-49	Ag(2.4 μg/l) Cd(2.0 μg/l)	NH_3-N(3.3 mg/l) Cr(26 μg/l) Cu(32 μg/l)	Se(21.2 μg/l)	Se(20 μg/l)			Ag(5 μg/l)	Ag(5 μg/l)
50-69	Cu(46 μg/l)	TOC(6.7 mg/l) COD(18 mg/l) Zn(162 μg/l) Fe(66 μg/l)			Se(90 μg/l) Ba(35 μg/l) Cr^{+3}(6 μg/l)	Ba(40 μg/l)		
70-84	COD(4 mg/l)		Hg(40 μg/l)	Hg(45 μg/l)	Ag(3 μg/l) Hg(1 μg/l)	Ag(5 μg/l) Hg(1 μg/l) Cr^{+3}(5 μg/l)		
85-100			Se(20 μg/l)†	Se(25 μg/l)†	Cu(12 μg/l) Cd(13 μg/l) Cr^{+6}(18 μg/l)	Cu(12 μg/l) Cr^{+6}(18 μg/l)		

† higher initial concentration

** increase

TABLE 9

Pollutant Removal by Wastewater Treatment Processes

(Noted in Per Cent Removal)

CONSTITUENT	SECONDARY TREATMENT (BIOLOGICAL)	CHEMICAL PRECIPITATION			ACTIVATED CARBON ADSORPTION	COMMENTS ON ACTIVATED CARBON	RESIDUAL LEVEL (µg/1)	GENERAL COMMENTS ON REMOVAL
		LIME	FERRIC CHLORIDE	ALUM				
Total Dissolved Solids	P	P	-	-	P	-	-	Generally increase TDS with treatment. Reverse Osmosis effective in removal.
Ammonia Nitrogen	VG	P	-	-	P	-	-	Bio nitrification most effective; Breakpoint Chlorination and Stripping Towers F to VG
Nitrate Nitrogen	VG	P	-	-	P to G	Depends on anerobic bioactivity	-	Bio denitrification most feasible
Phenol	P	P	-	-	P	Limited by driving force to about 1 mg/1	>1 µg/1	Treatment methods not effective in reducing Phenol to 1 µg/1 limit
Trace Organics	G	G	-	-	G	Removal depends on specific organics	≈5 mg/1 TOC and COD	Chlorinated Organics may be increased with Breakpoint Chlorination; Ammonia Stripping effective in Removing Volatile Refractory Organics
Arsenic (As)	P to F	P to G	G	-	P	Reacts with Sulfides	3 µg/1	Depends on influent level, pH, and Redox potential
Barium (Ba)	F	P to G	-	G	P to G	Due to highly soluble nature	>30 µg/1	Enhanced precipitation as Sulfate conc. increases
Boron (B)	P	P	-	-	P	-	>290 µg/1	Generally negligible
Cadmium (Cd)	P to G	F to VG	-	F	P to VG	Old Carbon better	2 µg/1	High removals due to precipitation of Sulfide and Hydroxide forms

Poor (P) - <30% Fair (F) - 30-60% Good (G) - 60-90% Very Good (VG) - >90%

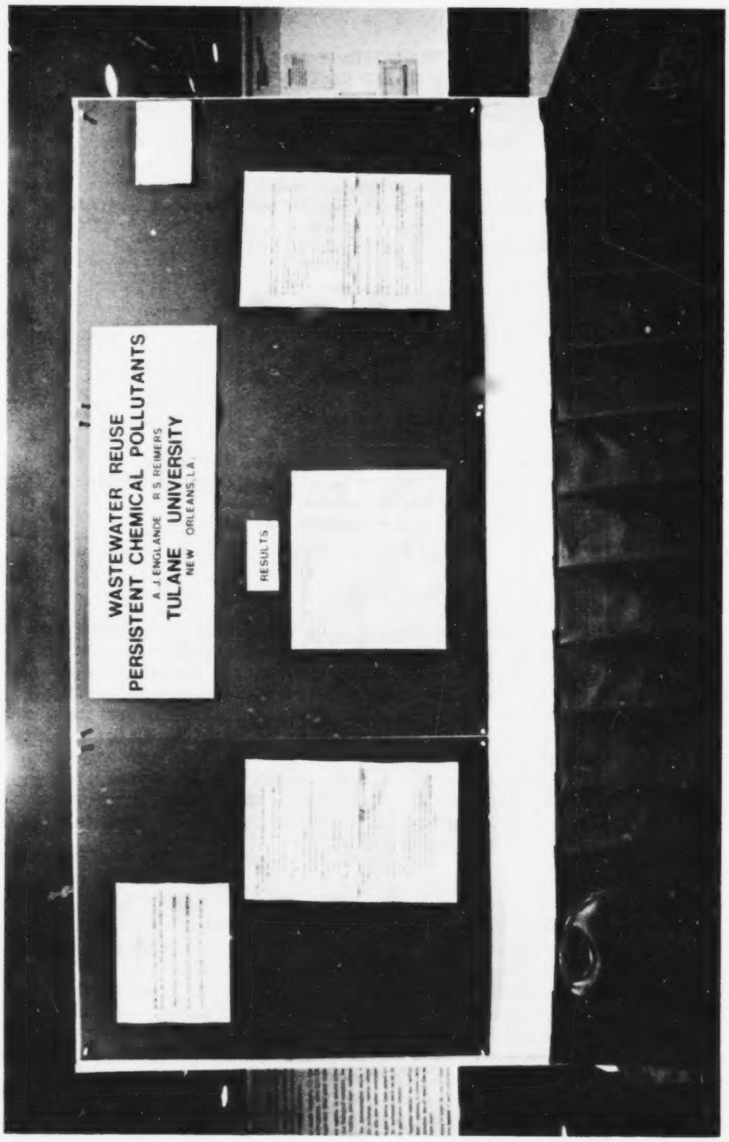
TABLE 9 Continued

Pollutant Removal by Wastewater Treatment Processes

(Noted in Per Cent Removal)

CONSTITUENT	SECONDARY TREATMENT (BIOLOGICAL)	CHEMICAL PRECIPITATION			ACTIVATED CARBON ADSORPTION	COMMENTS ON ACTIVATED CARBON	RESIDUAL LEVEL (μg/l)	GENERAL COMMENTS ON REMOVAL
		LIME	FERRIC CHLORIDE	ALUM				
Chromium (Cr)	F to G	G	VG	G (Cr^{+6}) VG (Cr^{+3})	P to G (Cr^{+3}) VG (Cr^{+6})	Reduction with Bio-activity Cr^{+3} less soluble than Cr^{+6}	20 μg/l	Depends on influent level and oxidation state
Copper (Cu)	P to G	P to G	-	G	G to VG	Enhanced Sorption better with New Carbon	70 μg/l	Influenced by Influent Concentration
Iron (Fe)	P to F	P to VG	-	-	P to G	Sulfide complexes ppt. but anaerobic bio-activity causes reduction to soluble Fe^{+2}	>40 μg/l	Depends on influent level, pH, and Redox potential
Lead (Pb)	F to G	F to G	-	VG	P to G	-	>5 μg/l	Enhanced precipitation with higher Sulfate levels
Manganese (Mn)	F	G to VG	-	P	P	Bioactivity on the Carbon reduces Mn^{+4} to Mn^{+2} and release	5 μg/l	Depends on pH and Redox potential
Mercury (Hg)	P to G	P to G	VG	G	P to G	Variability due to biological activity	5 μg/l	Removal is a function of pH, initial conc., and degree of complexation
Selenium (Se)	F	P to G	G	F	P to G	Variability due to highly soluble characteristics	2 μg/l	Depends on influent concentration
Silver (Ag)	P to G	P to VG	VG	VG	P to G	High affinity for Sulfhydryl groups	2 μg/l	Depends on influent level
Zinc (Zn)	F to G	P to F	-	P	P to G	Zinc Sulfide ppt.	>60 μg/l	Depends on influent and Sulfate levels

Poor (P) - <30% Fair (F) - 30-60% Good (G) - 60-90% Very Good (VG) - >90%


WASTEWATER REUSE
PERSISTENT CHEMICAL POLLUTANTS
A. J. ENGLANDE R. S. REIMERS
TULANE UNIVERSITY
NEW ORLEANS, LA.
RESULTS

SONICS AND ELECTROSTATICS - AN INNOVATIVE APPROACH TO WATER AND WASTE TREATMENT

Robert S. Reimers, III

Department of Environmental Health Science
Tulane University School of Public Health and Tropical Medicine
1430 Tulane Avenue
New Orleans, Louisiana 70112

Paul S. deKernion, III

Sonic-Clean, Inc.
Post Office Box 24004
New Orleans, Louisiana 70184

D. Blair Leftwich

Department of Environmental Health Science
Tulane University School of Public Health and Tropical Medicine
1430 Tulane Avenue
New Orleans, Louisiana 70112

Introduction

With the increased demand for water, any water treatment process requiring little to no chemicals is a conducive alternative for many industries. This is especially pertinent when the possibility of water reuse is considered. The application of an ultrasonic-electrostatic water treatment process has demonstrated reductions in operational and maintenance problems along with improvement of effluent water quality. The reuse potential for this process has been noted in the soft drink bottling industry, wastewater treatment plants, boiler water treatment, and cooling tower systems. The objective of this paper is to elucidate the effectiveness of this process in various industrial applications and its possible water reuse potential.

Background

The application of ultrasonics and electrostatics for water treatment, disinfection, wastewater treatment, and various industrial applications dates back to the Twenties (1,2), but as of 1978 there can be found no process utilizing both sonic and electromagnetic force fields in the literature or patent files. In general, past processes have used electrolytic treatment, induced electromagnetic force fields, and ultrasonics (3), yet there has been a great deal of skepticism concerning these low power water treatment processes which reduce chemical requirements (4,5,6). Recent research indicates that these low power processes indeed effect chemical requirements of water and there is a great deal of new scientific information demonstrating potential application in both water treatment and disinfection.

Between 1930 to 1950, there was a great deal of effort concerned with the conditioning of sludges by electrolytic and ultrasonic treatment. In summary, it appears that electrical treatment can be substituted for chemical conditioning for a wide variety of liquid sludges, if the power economics were the only major consideration. However, the sludge handling and equipment maintenance problems appeared to preclude this process until improved techniques are developed (1). In like manner, conditioning of sewage sludges by ultra or supersonic vibration has been explored in Europe (7,8). As of the early 1950's, the available data indicated that the application of ultrasonics was not successful in sludge treatment. Ultrasonic vibrations were noted to degasify sludges which is beneficial, but the vibrations also tended to destroy sludge flocs resulting in fine and charged solids that are more difficult to dewater.

The application of electromagnetic fields in water and waste treatment has received recent attention in the literature (2). Water purification processes using magnetic separation with a magnetic field of 700-20,000 gauss has been very successful in removing coliforms, turbidity, color, phosphate, and suspended solids from sewage sludge, separating oil, solids, and phenols from refinery wastes, polishing metal oxides, clays, and other ions from thermal power plant waters (boilers, cooling tower effluents, and etc.), and removing iron oxides, silicates, and oils from steel mill wastes. However, there are still many process and cost problems which must be overcome before the application of electromagnetic force fields are feasible, except under specific conditions.

Ultrasonics has been utilized in industry for over 50 years. The original applications were used by the French in 1919 for detecting submarines and killing fish. Ultrasonics is currently used in the textile industry (for dispersing pigments, spin finishing applications, wool creaming, and cotton warp preparation), in the pharmaceutical industry (for the manufacturing of emulsions, the despersion of solids, and in the development of antibiotic suspensions), in the cosmetic industry, in the chemical industry, etc. (9). Future application appear to be in filtration processes , control of chemical reactions, depolymerization and polymerization techniques, process control and heat transfers processes. As the influence of ultrasonics on biologics and chemical systems is better understood, the reduction of costly chemicals and high power demands can be resolved.

In 1865, the first water conditioning gadget patent in the United States was applied for and accepted. These gadget processes offer a something-for-nothing or a kind of panacea process. These gadgets were claimed to be special devices requiring no technical control and to treat water by nonchemical means to prevent scale, corrosion, and other problems encountered in the industrial use of water (4). In the late 1950's , Eliassen, et al (5) investigated various water conditioning processes utilizing induced magnetic fields and noted the following observations:

1) Water treated by magnetic fields did not inhibit scale formation.

2) The rate of corrosion was not influenced by magnetic fields over a 4 month period.

3) Conditioned waters used in accelerated laboratory corrosion experiments indicated no significant inhibition control.

A plausible reason why the magnetic conditions were not effective was
because of the low magnetic fields used as compared to the high magnetic
fields required (10,000-fold difference in electron densities). In 1977,
a publication warned the public of devices which claim to operate on
electromagnetic, electrostatic, magnetic, sonic, or any other physical
principles for the prevention of scale and corrosion. Generally, they
felt that these processes were in conflict with basic scientific laws,
allowed far too little exposure to the field, questioned the ability of
cations to adsorb electrons, and felt that this treatment would not be
carried beyond the treater (6). In most situations, the above skepticism
should be of major concern for all professionals, but recent research has
brought to the forefront data that indicates the application of electro-
statics and ultrasonics as a viable approach to water treatment.

In the last ten years, researchers have noted that magnetic and
electrostatic fields have been beneficial in the reduction of boiler
scale (especially the work of Russian researchers) (11-14). Toshi and
Kamat (11) have noted substantial pH drops in distilled water (0.62 pH
units) exposed to magnetic field of 1900-5700 gauss. Similar observations
have been reported for naturally occuring waters (13). Measurements of
magnetic effects on the physical properties of water have usually been
part of an investigation of the reduction on boiler scale which results
when the water supplied to a boiler is passed through a magnetic field.
A variety of explanations have been proposed for this phenomena, and one
such explanation involves a magnetically induced change in the pH of water
(11). The reduction in pH indicates an increase in the Lewis acids (elec-
tron donors) which in turn can carry electrons to scales. Quickenden,
et al (15) were not able to substantiate the above observations with dis-
tilled water, but this could be due to many other water quality factors.

In the last fifteen years, the influence of ultrasonics on aqueous
systems has been investigated by major research institutes in Russia
(Institute of Ultrasound), Germany, and Austria. The application of ultra-
sound has been found to decrease the solubility of mineral oils in water,
increase the formation of electron excited states in water, increase the
filtration of fluids through filters (water and oils), accelerate chemical
reactions (especially oxidation-reduction reactions), influence the effect
of catalytic reactions (either way), promote strong electron ionization
and polarization, and improve heat transfer (3,16,17). It is interesting
to note that many chemical reactions and molecules are influenced by
ultrasonics in unique ways. The oxidation of ferrous ion to ferric ion
in 0.8 normal solution of sulfurric acid occurs only in the presence of
ultrasonics and at an optimum frequency of 15-20 KHz (3). When investigat-
ing water treatment, the above phenomena can be of major importance. Another
important factor concerning the application of ultrasonication emphasized
by Murry (3) and Ouelette, et al (2) was that ultrasonication alone would
not have a germicidal effect or increase reactivity, but when used with other
treatment methods or catalysts, the influence of the ultrasonics can make
the reactions or biocidal magnitudally more effective.

Disinfection has been accomplished by the application of ultrasonics both in conjunction with chemical and heat, and/or without either (18-25). The disinfection of microbes with ultrasonication was first reported in 1929 (22). Recent investigation showing that this disinfection appears to be a function of intensity not frequency. For example, Esherichia coli B are inactivated only if the intensity is kept at 2.25 watts/cm^2 or greater (20). Similar observations have been noted by Croakley, et al (24). All say that ultrasonication will kill microbes, but will not give a 100% kill. Therefore, the problem of regrowth can become a major concern (21,23). Finally, ultrasonication can be used as a disinfection process when coupled with heating or chemicals, and in fact, the synergistics of heat and/or chemicals can make ultrasonication economically feasible (18,19,25).

Theory and Description of Process

The basic concept behind electrostatic treatment has been established for many years. The basic phenomenon follows the five (5) scientific concepts:

1) Matter is composed of atoms which contain outer valence electrons.

2) Minerals generally form ionic bonds which are specifically arranged into a definite structure or ionic lattice.

3) These ions within the crystalline lattice are held in place by the balance of both attractive and repulsive elctrostatic forces. Generally, ionic salts will dissolve in aqueous solutions due to the highly polar structure of the water molecules.

4) Immedicately prior to precipitaiton of any inorganic substance such as calcium sulfate, these ions will form a quasi-ionic lattice similar to its solid crystalline structure (commonly called liquid crystals).

5) A magnetic field resulting from an electrode will exert a force on an electron moving perpendicular to it. Therefore, an electrode will tend to dislocate the ionic lattice by displacing the electrons from their normal state and inhibit the formation of solid crystals.

Thus, the application of an electrode will induce an electromagnetic field which can inhibit crystalline growth and reduce scale adherence to the sides of metallic or other charged surfaces (12).

Even though the electrostatic treatment has met with some success, there are some major limitations. The water must not exceed a temperature of *degrees F. or a water pressure of 300 PSI. The application of the Aqua Scrubber Water Treater has some distinct advantages. The ultization of the ultrasonicator along with centrifugal hydraulics creates both sonic and hydraulic waves.

This sonic and hydraulic action causes an osmotic wave due to the increased conductivity of soluble salts, and hydraulic waves due to additional turbulence. The combination of these factors cause a shock wave

*(200°F)

which aids in physically dissolving and flaking of scale along with the possible destruction of bacteria. The use of sonics and hydraulics in conjunction with an induced electromagnetic field has a synergistic effect due to fluctuations in the osmotic pressure, the static pressure, ultrasonic force field, and electronic force field (9).

The specific process under discussion incorporates two (2) physical regions, the hydrofracture and an electron generator chamber as shown in Figure 1. In the hydrofracture region, the hydraulic flow is optimized, and the ultrasonication is applied to the aqueous solution throughout the whole system. Within the electron generator chamber, the water flows through an induced electrostatic force field. Due to this treatment, molecules are made unstable by adsorbing electrons. These, unstable compounds inhibit the formation of precipitates, disintegrate existing scale (to soluble salts and sludge*surries), reduce bioactivity and contamination, and convert soluble organics to less soluble detergents.

The hydrofracture chamber is divided into two (2) components, the centrifugal and ultrasonic systems. The centrifugal hydraulics performs the following functions:

1) Increases the exposure of water to the subsequent force fields.
2) Removes gases and reduced cationic scale from the electrode (which could decrease the performance).

The ultrasonic system increases the process efficiency by the following factors:

1) Breaks up and agitates water, minerals, and organics enabling superior water treatment.
2) Generates kinetic energy which excites millions of atoms and molecules.

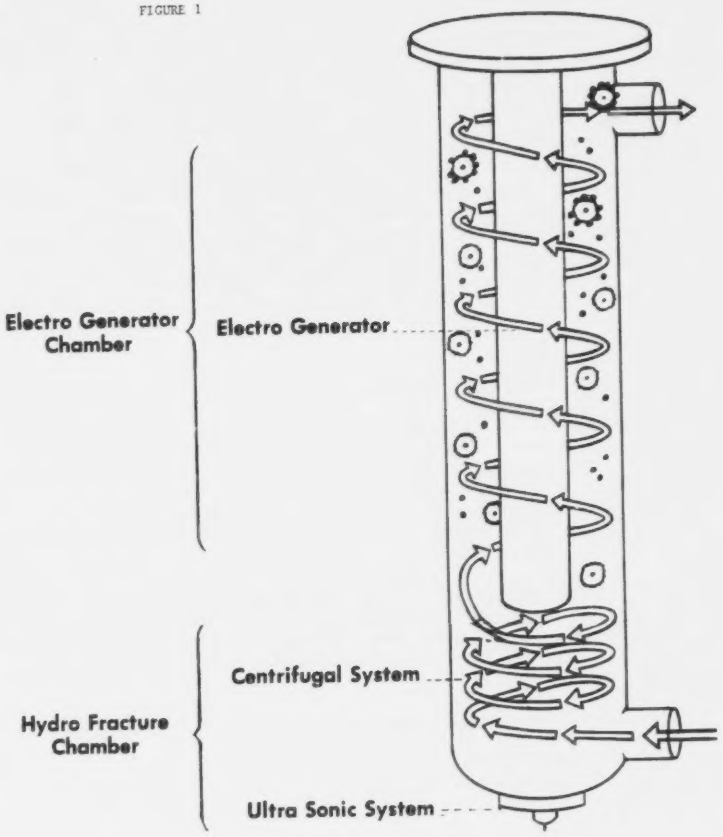
By the above two (2) mechanisms, the effectiveness of the electromagnetic force field is greatly increased.

In the electron generator chamber, the previously treated water is exposed to the electrostatic force field. This force field is generated by an electrode releasing millions of electrons into the water flowing by the electrode. The negatively charged electrons are **adsorbed** by the dissolved constituents (molecules and atoms which act as electron carriers). The activated carriers release their electrons to the ionic bonds of crystalline scale, the scale will either redissolve into the water or form a sludge slurry.

The combining of the electron with a cationic salt will greatly reduce mineral's ability to scale. This process inhibits the mineral's ability to form crystalline structures, and by the same process, crystalline foundations which already exist will disintegrate or will form amorphous like structures (slurry) instead of clearly developed crystals. The above inhibition and/or disintegration of scale is due to the reaction

*slurries

FIGURE 1


Electro Generator Chamber
Electro Generator
Centrifugal System
Hydro Fracture Chamber
Ultra Sonic System

of the electrons.

Substantiation of the previously discussed theory is described in this section. Analysis of the initial parameters over a period of minutes, indicates little to no treatment as shown in Table 1, with the exception of the adhesion of residue to the container and to its location. This information indicates that the treated water has an influence on the structural and adhesive properties of the inorganic precipitates. However, when one looks at the influence of these parameters over the period of a week, there are some very noticeable changes in water quality. After treating a 10,000 mg/l solution of Na_2SO_4 (60 1) continuously for more than a week, the pH, oxidation-reduction potential, and alkalinity were observed to drop from 10.0 to 8.5, 104 to 42 millivolts and 136 to 106 mg/l, respectively. This data validates the premise that electrons are absorbed by the soluble constituents forming Lewis acids which should lower the pH, oxidation-reduction potential, and alkalinity. As further varification that there is some influence of the sonication and electrostatic force fields, samples passed through the aqua scrubber (ultrasonic-electrostatic water treater) from a secondary aeration basin and secondary clarifier were found to have higher BOD's, fecal coliforms, and bioactivity in its solids. This phenomena is thought to be due to the sonics solubilizing the BOD and microbes from the suspended solids which inturn increased the resulting sludges bioactivity.

Materials and Methods

The data presented in this paper was obtained from several different industrial application such as soft drink bottle industry, cooling tower application, etc. All samples collected were analyzed according to the procedures in "Standard Methods" (26) with modification to these when necessary. These modifications are described in the "Manual for DR-EL/2" by Hach Chemial Company (27) and a manual on conductivity meter (Cambridge Scientific Industries, Inc.) (28).

Data for the bottle rinsing machine was obtained from samples collected at the discharge pipe from the rinser over a period of three and one-half months. These samples were measured for total solids (T.S.), suspended solids (S.S.), total dissolved solids (T.D.S.), pH, conductivity, temperature, chlorides, calcium, and alkalinity.

Conductivity, pH, and temperature were measured at time of sampling where practicable or within one hour after sampling.

The cooling tower data obtained during an eight-month period. Parameters measured were pH, temperature, conductivity, hardness, alkalinity, T.S., S.S., and T.D.S.

Boiler water (blowdown) samples were measured for pH, temperature, conductivity, hardness, sulfate, alkalinity, chloride, and iron. Samples were drawn from the blowdown outlet one minute after opening the blowdown valve.

Domestic wastewater samples were analyzed for BOD_5, pH, temperature,

TABLE 1. Influence of the Treatment on the Water

CONSTITUENT	UNITS	INFLUENT	EFFLUENT	INFLUENT	EFFLUENT
Specific Gravity at 73°C		1.00	1.00		
pH at 23°C	(S.U.)	6.20	6.20	7.7	7.7
Total Suspended Solids	(mg/1)	14.0	14.0	6.6	6.6
Total Dissolved Solids	(mg/1)	-	-	219.0	219.0
Phenol. Alkalinity	(mg/1 as $CaCO_3$)	0.0	0.0		
Total Alkalinity	(mg/1 as $CaCO_3$)	16.0	16.0		
Total Hardness	(mg/1 as $CaCO_3$)	261.0	263.0	216.0	216.0
Sodium	(mg/1 as Na)	195.0	205.0		
Potassium	(mg/1 as K)	39.0	40.0		
Calcium	(mg/1 as Ca)	68.0	66.0		
Magnesium	(mg/1 as Mg)	22.0	24.0		
Iron*	(mg/1 as Fe)	6[2]	7[2]		
Chloride	(mg/1 as Cl)	152.0	157.0		
Sulfate	(mg/1 as SO_4)	80.0	72.0		
Bicarbonate	(mg/1 as HCO_3)	19.0	19.0		
Carbonate	(mg/1 as CO_3)	0.0	0.0		
Phosphate	(mg/1 as PO_4)	22.0	23.0		
Nitrate	(mg/1 as NO_3)	929.0	969.0		
Conductivity at 73°F	(ohm/m)	6.45	6.45		
Solids Residue				Poor	Good
Adhesion of Residue to Container				Strong	Weak
Location of Residue in Container				Bottom and Sides	Bottom

*[ ] - Soluble

dissolved oxygen (D.O.), suspended solids (S.S.), volatile suspended solids (V.S.S.), flow rates, fecal coliforn, settleable solids, sludge volume index (S.V.I.), mixed liquid suspended solids (M.L.S.S.), and sludge removal. Fecal coliform counts were obtained using the membrane filter procedure in "Standard Methods" (26). Temperature, pH, D.O., and BOD_5 measurements were made at time of sampling.

Results and Discussion

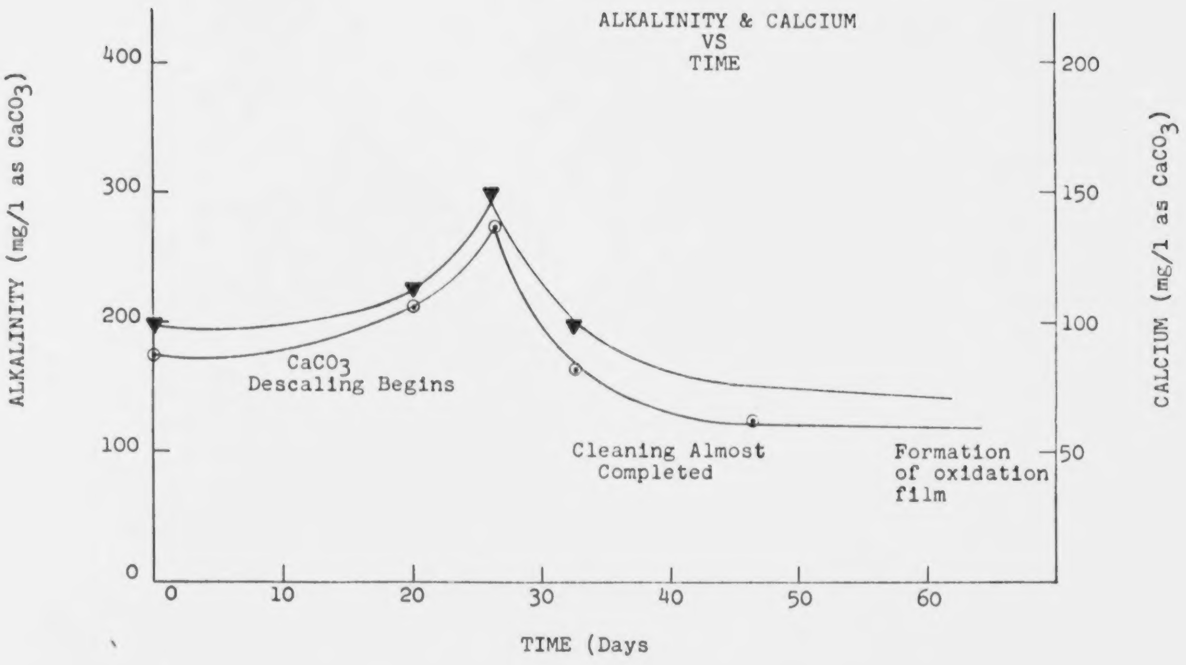
From field testing, the application of this water treatment instrument (The Aqua Scrubber) in the soft drink bottling industry has reduced the operation and maintenance problems to a minimum and decreased possible noxious effluents. A clogged bottle rinsing machine was descaled within three months and the resulting effluent reduced to the quality of the influent tap water as shown in Figures 2 and 3. By this process, the machinery did not require acid washing. This effluent water as illustrated in Table 1 could be reused for other application, such as, influent water for the bottle warming process, cooling tower influent, etc. An old bottle warming machine was completely clogged and releasing a septic black effluent. This effluent had a high BOD (> 200 mg/l), a high level of bioactivity, and very high soluble salts content (29). After one month's treatment, large quantities of calcium sulfate were flaking off the bottle warmer; the BOD was reduced to less than 25 mg/l; and coliform counts were reduced to less than 10 coliform per 100 ml. as shown in Table 2. Therefore, this instrument improved the effluent quality so that the resulting sewerage surcharge could be reduced by one magnitude. For better water management, an aqua scrubber was placed on line to a new bottle warming machine. The warmer's retention tank was discharged weekly in a batch basis to the sewer. The warmer was monitored for biological parameters over the next four weeks under four different operating conditions. These conditions are listed below:

1) Scrubber was turned completely off (control).
2) Scrubber was on for only eight hours daily.
3) Scrubber was on for 24 hours daily.
4) Scrubber was on for 24 hours daily along with a 20% standard dosage of phosphoric acid and methyliodide, (the biocide normally used to reduce biological problems).

The data compiled from this monitoring is listed in Table 3. From the results shown in Figures 4 and 5, the minimum concentrations for BOD_5 and total bacterial counts appears to be every 2-1/3 days. Amazingly, the application of the disinfectant appeared to have little influence and even reduced the effectiveness of this treater. This phenomena can possible be attributed to two factors: (1) the photodegradability of the methyliodide and (2) the inhibition of treatment resulting from the methyliodide. Therefore, the apparent water management program is to treat the bottle rinser water and reuse the rinser discharge for the bottle warmer influent, boiler water feed, and/or cooling tower water. Before its second use, this water should again be treated by this scrubber, and its final disposal will depend on its use. The bottle warmer water should be discharged within 2-1/3 days or after approximately 20,000 bottles have passed through warmer. Cooling tower or boiler operations should be discharged depending upon optimum blowdown frequency.

BOTTLE RINSING OPERATIONS (COOL WATER)

FIGURE 2



BOTTLE RINSING OPERATIONS (COOL WATER)

FIGURE 3

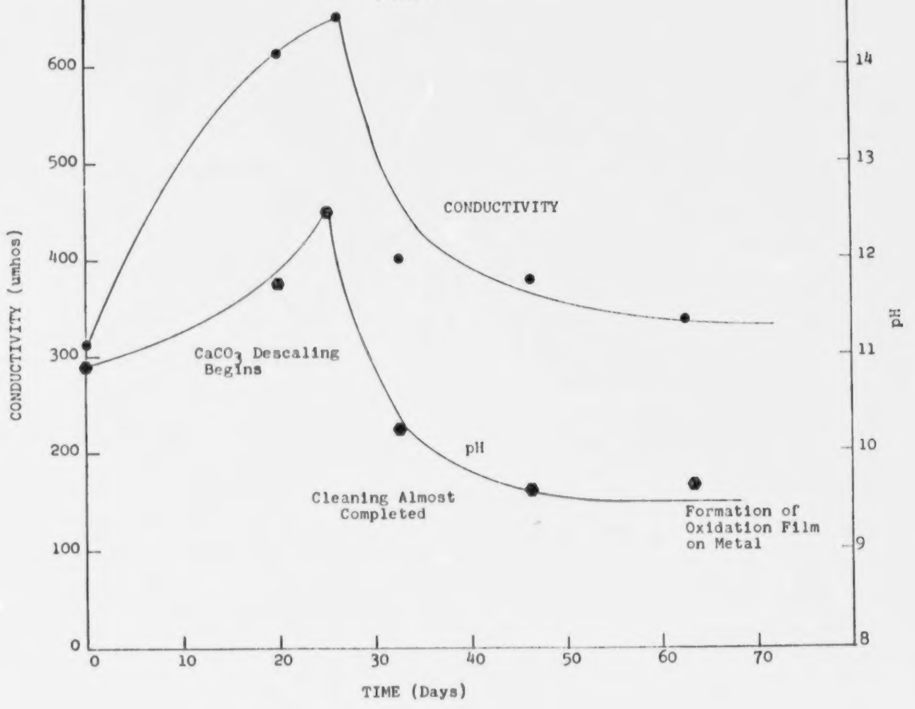


TABLE 2. Bottle Rinsing Effluent During Descaling Process

TIME (days)	Ca^{+2} (mg/1)	Alk. (mg/1)	Cl^- (mg/1)	T.D.S. (mg/1)	S.S. (mg/1)	T.S. (mg/1)	pH	Temp. (°F)	Cond. (μmhos)	Hard. (mg/1)
0	100	175	50	200	2	202	-	59	-	-
20	110	215	50	220	5	225	-	59	610	-
26	150	270	75	420	9	429	12.5	59	650	-
32	100	160	30	317	1	318	10.2	58	400	-
36	110	125	50	0	0	280	9.6	57	380	-
65	120	135	-	422	2	424	-	48	340	-
75	-	-	-	306	3	309	10.1	62	500	-

TABLE 3. Bottle Warming Effluent During Descaling Process

DAYS AFTER START-UP	pH	Alk. (mg/1)	Temp. (°F*)	Cond. (μmhos)	Hard. (mg/1)	T.D.S. (mg/1)	S.S. (mg/1)	Ca^+ (mg/1)	Odor	Color
0	8.5	115	94	400	140	301	29	-	very strong	black
5	7.5	120	80	400	196	252	44	20	strong	brown-yellow
7	7.6	115	102	500	236	366	36	18		
14	8.2	115	106	400	-	277	17	-	mild	yellow
18	7.1	120	104	450	236	496	29	18		
19	7.3	-	114	500	220	346	26	20	weak	straw
25	7.7	115	104	220	170	-	-	30		
28	7.2	115	94	350	130	-	-	25	trace	clear

*Drain water temperatures (from tank)

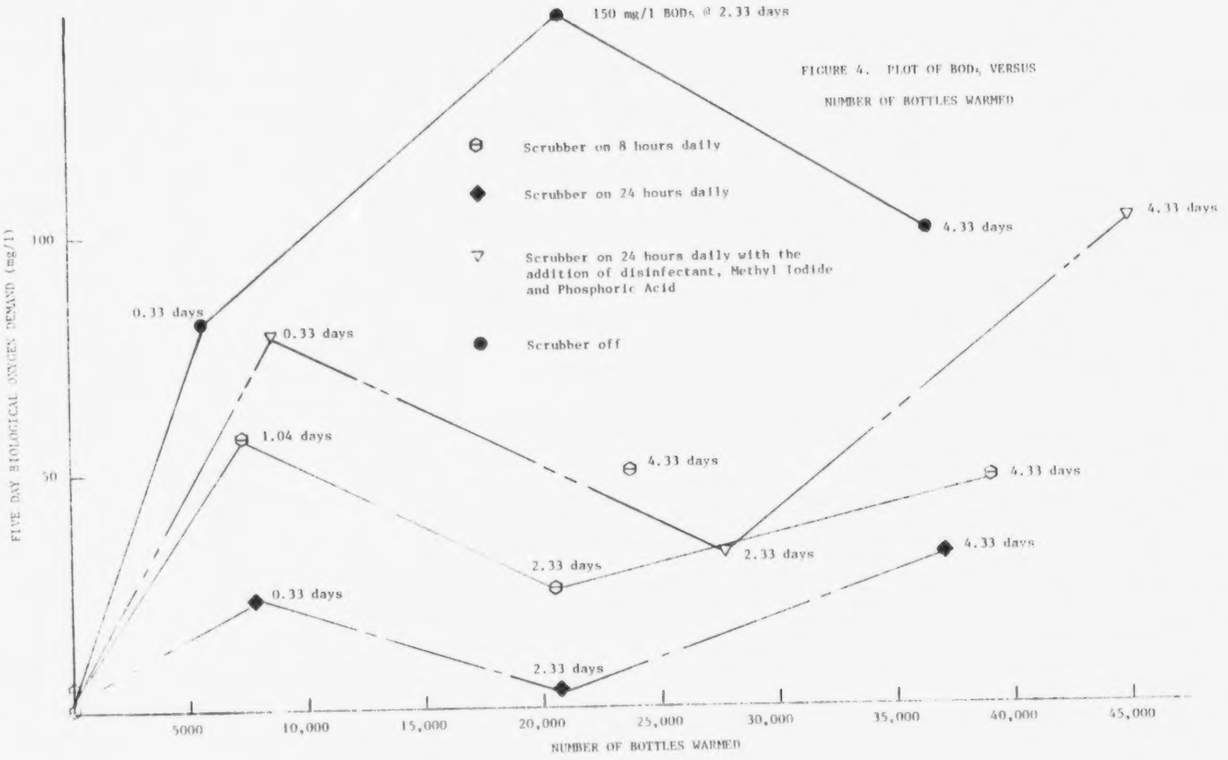


FIGURE 4. PLOT OF BOD$_5$ VERSUS NUMBER OF BOTTLES WARMED

FIGURE 5. PLOT OF LOG OF NUMBER OF BACTERIA VERSUS THE NUMBER OF BOTTLES WARMED

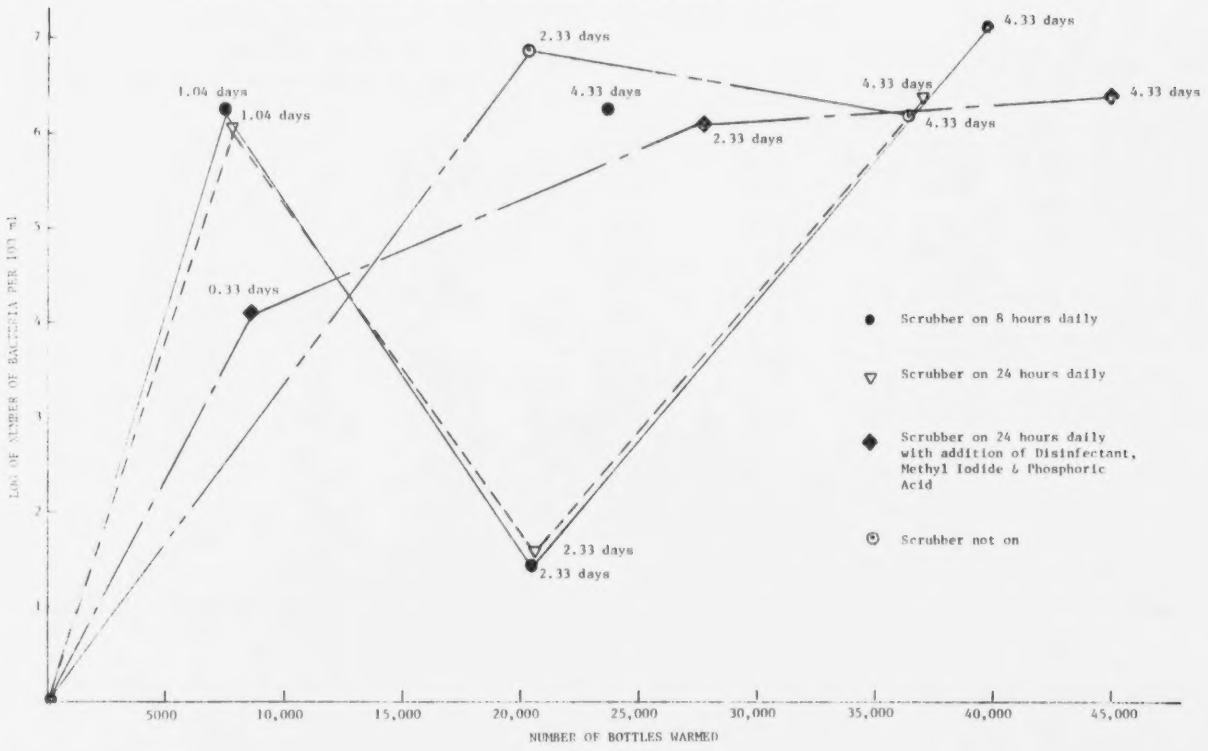


TABLE 4. Bottle Warming Machine's Water at 140°C

Time (days)	pH (S.U.)	BOD_5 (mg/1)	TOC (mg/1)	Total Bacteria (#/100 ml)	Suspended Solids (mg/1)	Volatile Suspended Solids (mg/1)	Number of Bottles	Comments
0.00	9.8	5	44	1	1.0	1.0	0	Scrubber was
0.33	7.7	81	58	99	3.0	3.0	5,608	turned off
1.33	8.1	63	68	9	7.0	7.0	11,082	"
0.00	9.7	0	48	0	0.5	0.5	0	"
2.33	7.5	150	84	$7x10^6$	11.0	9.5	20,503	"
4.33	7.5	90	111	$2x10^6$	17.5	14.5	36,428	
1.04*	7.5	57	61	$1.7x10^6$	5.0	5.0	7,330	Scrubber on
2.33*	8.2	25	58	25	11.0	9.5	20,503	8 hours daily
4.33	7.2	48	92	$13.2x10^6$	12.5	12.5	39,082	"
4.33	7.7	50	130	$1.7x10^6$	13.0	13.0	23,917	"
0.33	8.1	32	55	132	3.0	2.0	8,081	Scrubber on
2.33	7.2	33	88	$10.8x10^6$	11.5	11.5	21,893	24 hours daily
1.13	7.5	23	51	$1.1x10^6$	7.0	7.0	7,786	"
2.13	9.7	3	38	35	2.0	2.0	20,766	"
4.33	7.4	32	63	$2.3x10^6$	16.0	14.0	37,071	"
0.33	7.6	79	58	$1.2x10^4$	4.0	3.0	8,619	Scrubber on**
2.33	7.3	32	89	$1.2x10^6$	44.0	39.0	27,809	24 hours daily
4.33	7.0	104	134	$2.3x10^6$	77.0	76.0	44,988	with oil disinfection

* Monday-Wednesday temperature = 60-80°C

** Disinfectant was phosphoric acid and methyl iodide

From various field tests, the Aqua Scrubber has been applied to both one-through and recirculating cooling towers. In both cases, the cooling towers were scaled up, and over the period of 120 days of treatment, there was considerable loss of scale noted by high suspended solids, increase in conductivity readings, and greater salt concentrations. In fact, the soluble salts were found to be one magnitude above their soluble limits and then there was a drop in conductivity and salts after the scale had been removed as shown in Figure 6. Due to the high salt levels, the recirculating cooling tower water was not reusable, but the once-through cooling tower effluents appear amenable to irrigation, etc. This is due to the lower disinfectant requirements, elimination of the need for descaling agents, little or no need for anti-corrosive agents, such as zinc or chromium, and a reduction of soluble salt levels to less than 300 mg/l. Typical effluents are shown in Table 5.

In the investigation of boiler water treatment, the application of this treater caused the scale to fall off the walls and pipes of boilers. For the field-monitored boilers, the conductivity of the blowdown water increased immediately to 7000 µmhos, and depending upon the scale or the frequency of blowdown, the conductivity has been noted to rise to 11,000 µmhos for a sulfate scale. The scale began to drop off the walls at 40 days and after 80 days the system was completely cleaned. After 120 days a protective oxide film had formed. Other parameters which simulated the conductivity for tube-fired boilers were hardness, dissolved solids, calcium and sulfate, but obviously the water quality parameters which are affected depend on the specific scale. These results are illustrated in Figure 7. Obviously, the direct reuse potential of this boiler water will be very limited, but by using this treater, there will be a lower water demand and the resulting blowdown water sill have less chemical additives which would limit discharge or require pretreatment.

Concurrently, preliminary research is being conducted in the field on the treatment of secondary effluents. The application of this process was found to reduce the BOD by fifty percent and total coliform by seventy percent as a function of both flow and initial concentration as illustrated in Figures 8 and 9. Before this process could be considered as a viable tertiary treatment system, the effects of temperature, suspended solids, and acclimatized biogrowth must be ascertained and resolved. Even so, this process has the potential for reducing the oxygen demand of a secondary effluent by fifty percent and the chlorine demand considerably. This resulting effluent as shown in Table 6 could easily be used for irrigation water. There has been some speculation that ultrasonically and electrostatically treated domestic waste water may actually induce greater plant growth. It should be emphasized that the treater is designed presently to descale, and even though there is a seventy percent kill of domestic waste microbes, there will be a regrowth within one hour if there is not immediate disinfection. At one treatment plant where pilot tests are currently being conducted, the chlorine requirement has been reduced by at least 20 percent (30).

The effects of the Aqua Scrubber on the secondary waste treatment process proved interesting. Two process configuration alternatives were: (1) Treatment of the effluent from the aeration basin; and (2) Treatment of the partially settled secondary clarifier water. The greatest influence appears to be in the treatment of the aeration basin effluent which would

FIGURE 6

Plot of Conductivity Versus Time
For a Recirculating Cooling Tower

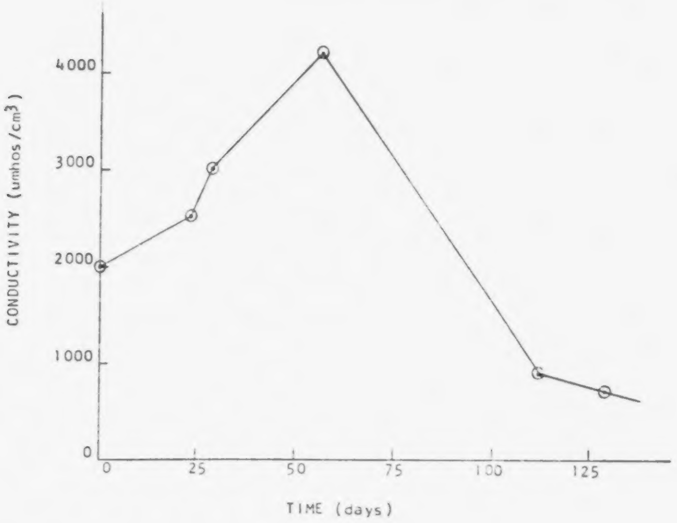


TABLE 5. Once Through Cooling Tower Effluent

DAYS	pH	TEMP. °F	COND. µmhos	T.D.S. mg/1	S.S. mg/1	T.S. mg/1	HARD. mg/1	ALK. mg/1	Ca# mg/1	Fe. mg/1	SO4 mg/1
0	-	58	300	138	6	144	-	85	-	0.08	5.0
6	6.5	44	475	380	11	391	-	130	0	1.0	2.0
16	8.6	82	570	441	3	444	-	145	-	-	-
23	8.7	46	910	632	2	634	-	205	-	-	-
64	8.7	80	350	-	2	-	40	110	20	-	-
72	8.7	82	375	261	2	263	160	110	20	-	-
77	8.7	60	350	263	1	264	152	110	20	-	-
97	7.8	64	360	-	-	-	151	100	-	-	-
104	8.2	84	450	-	-	-	180	115	-	-	-
125	8.5	82	450	-	-	-	190	140	-	-	-
138	8.8	90	570	-	-	-	-	-	-	-	-
147	9.1	89	520	-	-	-	240	190	-	-	-
154	9.3	90	530	-	-	-	SAMPLE	LOST	-	-	-
161	8.6	90	450	-	-	-	233	180	-	-	-
167	8.7	94	660	-	-	-	333	220	-	-	-
173	9.1	90	780	-	-	-	350	202	-	-	-
202	-	92	820	-	-	-	300	250	-	-	-
242	8.7	90	640	-	-	-	240	190	-	-	-

Figure 7
Plot of Conductivity and Hardness Versus
Time For a Tube Fired Boiler

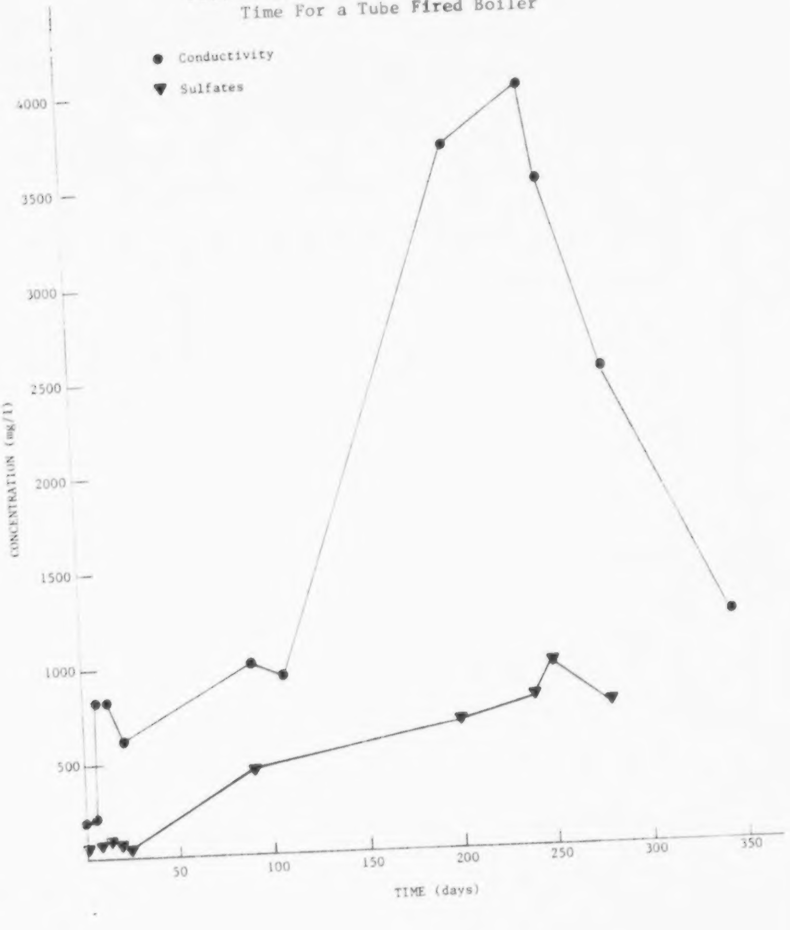


FIGURE 8. PLOT OF PER CENT BOD_5 REDUCTION VS. FLOW IN GPM

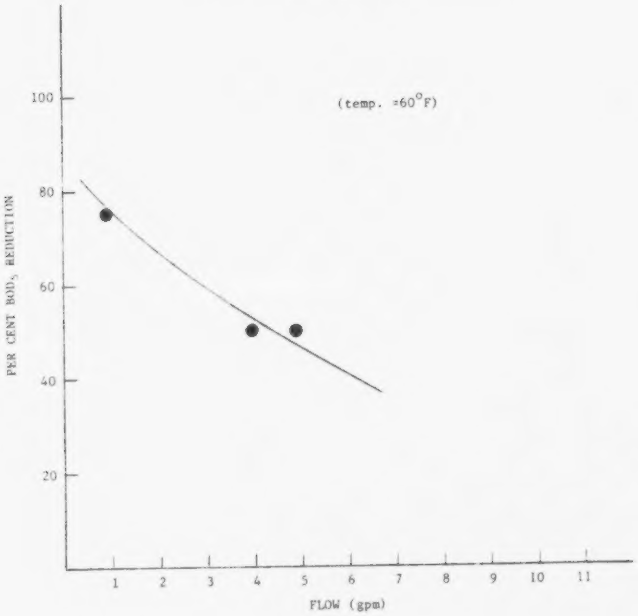


FIGURE 9. PLOT OF BOD_5 AND TOTAL COLIFORM INITIAL CONCENTRATIONS VERSUS PER CENT REMOVAL

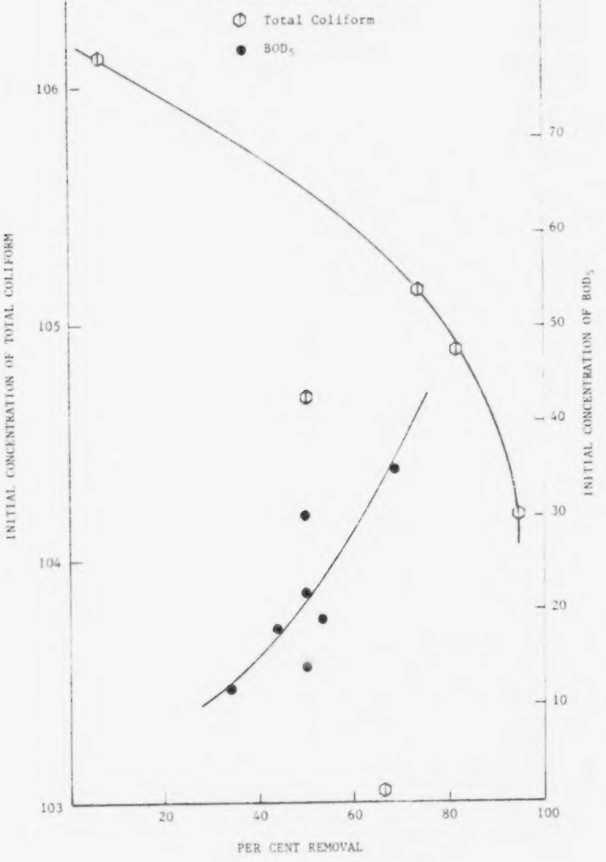


TABLE 6. Scrubber Treatment of Secondary Effluents

Time (days)	Flow (gpm)	pH (S.U.)	Temp. (^oF)	D.O. (mg/1)	SS(VS) (mg/1)	BOD Influent (mg/1)	BOD Effluent (mg/1)	BOD % Removal	Coliforms Influent (Total/100 ml)	Coliforms Effluent (Total/100 ml)	Coliforms % Removal
Kev 6.1											
1	5	7.4	57	8.9	16(13)	--	--	--	--	--	--
2	5	7.3	60	9.6	19(13)	30	15	50.0	--	--	--
3	5	7.1	60	9.0	8 (8)	--	--	--	--	--	--
4	5	7.1	60	8.8	--	--	--	--	--	--	--
5	5	7.2	62	8.7	13(11)	63	27	57.0	--	--	--
6	5	7.0	62	8.7	17(12)	--	--	--	--	--	--
7	5	7.2	62	8.8	--	35	11	69.0	--	--	--
8	5	7.2	60	9.1	5 (5)	22	11	50.0	--	--	--
9	5	--	60	8.7	5 (5)	19	9	53.0	9.5×10^5	1.7×10^5	82
Kev 6.1											
7	1	7.3	80	--	--	18	17	6.0			
11	1	7.3	80	--	--	23	25	0.0	Biological Feedback Problem		
14	1	7.3	66	--	--	35	39	0.0			
Kev 6.1											
1	4	7.1	70	--	--	14	7	50.0	$>1.6 \times 10^4$	1.6×10^4	--
Kev 8.1											
1	7.5	7.1	70	--	--	12	8	33.3	3.5×10^4	1.6×10^3	95.4
8	7.5	7.5	77	--	--	18	10	44.4	1.3×10^6	1.2×10^6	7.7

obviously have the highest solids (around 2000 mg/l of MLSS). The final effluent showed the following changes: a lower pH (<0.3 units), higher suspended solids (doubled), higher BOD's (over double), increased fecal coliform (by a factor of 200), and increased caustic requirement. Another unusual point was the doubling of the sludge volume to be removed. The treated clarifier water exhibited similar, but generally less, effects except for the released fecal coliform and sludge volume removed. These results are listed in Table 7. An explanation for this observation is that the ultrasonics and electrostatic force fields solubilized degradable carbon, converted non-degradable carbon to degradable, and solubilized bacteria associated with the solids. Therefore, there was an increase in the return sludges bioactivity which caused an increase in the quantity of organic acid produced. This increase in acid caused the dramatic drop in pH in the aeration basin. This treater appears to influence the microbes and organics and has a viable application in sludge treatment.

Conclusions

This innovative water treatment process has demonstrated various water reuse potentials (thereby providing better water management). In the soft drink bottling industry, the application of reuse is feasible. The Aqua Scrubber-treated water coming from the bottle rinser may be utilized for the bottle warmer process, cooling tower applications, and/or boiler use. This final effluent water under proper management has the potential to reduce the bottle industries surcharge over one order of magnitude. By using this scrubber, the use of toxic biocides, descaling agents, and anti-corrosive additives for cooling towers can be greatly reduced. In the case of once-through cooling towers, their effluents may be potentially used as irrigation waters along with other possible in-plant usage. Even though the reuse possibilities are minimal for boiler waters, the Aqua Scrubber should reduce the water requirement for boiler blowdown with the additional decrease of chemical additives. Currently, preliminary research on the Aqua Scrubber use as an inexpensive tertiary treatment process may lead to the further utilization of this effluent water for irrigation due to lower BOD's, coliform levels, and lower quantities of toxic chlorinated organics. Finally, this process has a definite effect on the biological components and suspended solids in domestic wastewaters, but the possible applicaitons have not been ascertained at present.

Acknowledgments

The authors gratefully acknowledge the support of Tulane University and Sonic-Clean, Inc. through the auspices of Robert J. Gaffney and David C. Rabbit, President and Vice-President of Sonic-Clean, Inc. respectively. The cooperation and aid of Avondale Shipyards (especially Mr. Frank R. Robideau) and Coca Cola of Louisiana (especially Marcelino Malic) has enabled the authors to complete this work.

TABLE 7. Effect of the Aqua Scrubber on Secondary Treatment Process*

System	Effluent				Aeration Basin	Sludge			Neutralization Requirement	Flow
	BOD_5 (mg/1)	TSS (mg/1)	FC (#/100ml)	pH (S.U.)	MLSS (mg/1)	Settable Solids (ml/1)	SVI	Sludge*** Removed (gal/day)	(lbs soda ash/day)	(gal/day)
Standard Process	12 (6)	25 (10)	0.1 (0.2)	6.4 (0.6)	2900 (610)	173 (50)	63 (20)	860 (150)	0 -	23,700 (10,700)
Scrubber Treatment of Aeration Basin Effluent	29 (8.6)	57 (21)	20 (24)	6.1 (0.6)	1850 (1000)	135 (20)	82 (34)	1900 -	7.6 -	28,300 (10,600)
Scrubber Treatment of Recycled Secondary Clarifier Water	25 (1)	41 (37)	600 (840)	6.4 (0.3)	2280 (220)	95 (50)	43 (25)	2400** -	3.4 -	25,600 (21,000)

* ( ) = Standard Deviation

** May still be influenced by the effects of Configuration #1

*** Assumed solids content approximately 1.5%

References

1. Burd, R.S., "A study of Sludge Handling and Disposal", U.S. Department of the Interior, Federal Water Pollution Control Administration, Federal Water Pollution Control Administration, Publication WP-20-4, pp. 326 (May, 1968).

2. Ouellette, R.P., King, J.S., and Cheremisinoff, P.N., *Electrotechnology Volume 1 - Wastewater Treatment and Separation Methods*, Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan, pp. 609 (1978).

3. Murry, E.J., "An Ultrasound Future or a Sound Sleep Part III Uses in Variety", *Chemical Technology*, *Vol. 5*, pp. 376-381 (June, 1975).

4. Welder, B.Q., and Partridge, E.P., "Practical Performance of Water-Conditioning Gadgets", *Industrial and Engineering Chemistry*, *Vol. 46,(5)*, pp. 954-960 (May, 1954).

5. Eliassen, R., Skrinde, R.T., and Davis, W.B., "Experimental Performance of 'Miracle' Water Conditioners", *Journal American Water Works Association*, *Vol. 50*, pp. 1371-1384 (Oct., 1958).

6. Dearborn Chemical Division of Chemed Corporation, "Watch Out For a New Round of Water Treatment 'Gadgets'", *Water Conditioning*, (July, 1977).

7. Husmann, W., "Effect of Supersonics in Sewage and Sludge by the McDonald Process", *Gesundh Ing.*, *Vol. 73* (718), pp. 127 (1952).

8. Coackley, P., "Research on Sewage Sludge Carried Out in the Civil Engineering Department of University College", *Journal Inst. of Sew. Purif.*, *Part I*, pp. 59-72 (1955).

9. Murry, E., "An Ultrasound Future or a Sound Sleep -Part I- Mechanisms", *Chemical Technology*, *Vol. 5*, pp. 108-111 (Feb., 1975).

10. Murry, E., "An Ultrsound Future or a Sound Sleep -Part II- Large-Scale Applications of Ultrasonics", *Chemical Technology*, Vol. 5, pp. 232-234 (April, 1975).

11. Joshi, K.M. and Kamat, P.V., "The Effect of Magnetic Fields on the pH of Water", *J. Indiana Chem. Soc.*, pp. 43 (1966).

12. Skorobogator, V.I., "Mechanism of the Action of Ultrasound, Magnetic, and Electrical Fields on a Scale Forming Liquid", *Izv. Vyssh Ucheb Zared*, *Energy*, *Vol. 13 (5)*, pp. 58-62 (1970).

13. Lazorenko, B.R. and Bautysh, L.A., "Properties of Water Subjected to the Action of Electrical of Magnetic Fields", *Elektron Obrab. Mater.*, *No. 3*, pp. 46-48 (1970).

14. Kirgintsev, A.N., "Mechanism of the Magnetic Treatment of Liquids", *Zh. Fiz. Khim.*, *Vol. 45(4)*, pp. 857-859 (1971).

15. Quickenden, T.L., Betts, D.M., Cole, B., and Noble, M., "The Effect of Magnetifc Fields on the pH of Water", The Journal of Physical Chemistry, Vol. 75(18), pp. 2830-2831 (1971).

16. Zharavler, A.I. and Tsvylev, O.P., "Formation of Electron States in Water on Exposure to Low Therapeutic (Pseudocavitation) Intensities of Ultrasound", Biofizika, Vol. 13(6), pp. 1018-1024 (1968).

17. Belcher, R.S. and O'Brien, E.J., "A modified Extraction Method for Determination of Mineral Oil in Sea Water", Bulletin of Environmental Contamination and Toxicology, Vol. 10(3), pp. 170-171 (1973).

18. Sierra, G. and Beucher, R.M., "Ultrasonic Synergistic Effects in Liquid-Phase Chemical Sterilization", Applied Microbiology, Vol. 22(21), pp. 160-164 (1971).

19. Ordonez, J.A. and Burgos, J., "Effect of Ultrasonic Waves on the Heat Resistance Bacillus Spores", Applied and Environmental Microbiology, Vol. 32(1), pp. 183-184 (July, 1976).

20. Dakubu, S., "Cell Inactivation by Ultrasound", Biotechnology and Bioengineering Vol. 18, pp. 465-471 (1976).

21. Stumpg, P.K., Green, D.E., and Smith, F.W., "Ultrasonic Disintegration as a Method of Extracting Bacterial Exzymes", J. Bacteriol., Vol. 51(4), pp. 487-493 (1946).

22. Schmitt, F.O. and Uhlemeyer, B., "The Mechanism of Lethal Effects of Ultrasonic Radiation", Proc. Soc. Exptl. Biol. Med., Vol. 27(7), pp. 626-628 (1930).

23. Ballanger, M.E., Brasher, W.J., and Maupin, C.C., "Water Contamination and the Ultrasonic Scaler", Va. Dent. J., Vol. 53(3), pp. 10-14 (Oct. 1976).

24. Coakley, W.T., Bater, A.J., and Lloyd, D., "Disruption of Micro-organisms", Advances in Microbial Physiology edited by Rose, A.H. and Tempest, D.W., Vol. 16, Academic Press, N.Y., pp. 279-341 (1977).

25. Wang, L.K., Wang, M.H., and Peery, G.G., "General Theories of Chemical Disinfection and Sterilization of Sludge - Part 2", Water and Sewage Works, Vol. 125(8), pp. 58-63 (August, 1978).

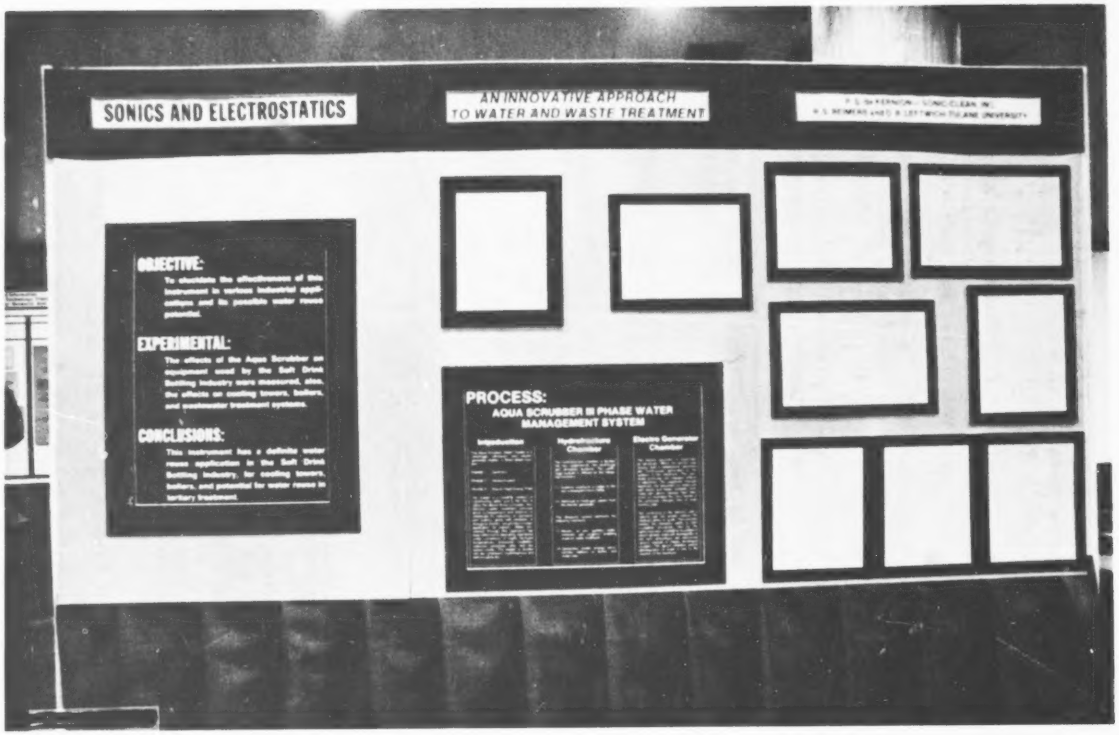
26. Standard Methods for the Examination of Water and Wastewater, 14th edition, American Public Health Association, Washington, D.C., pp. 1193, (1976).

27. Hack Chemical Company, Manual for DR-EL/2, Loveland, Colorado (1975).

28. Cambridge Scientific Industries, Inc., Manual for HyDAC Portable Conductivity Meter, Cambridge, Maryland (1977).

29. Nemerouw, N.C., Theories and Practices of Industrial Waste Treatment, Addison-Wesley Publishing Company, Inc., Reading, Massachusetts, pp. 557 (1963).

30. Perez, T., Personal Communication, (March, 1979).



SONICS AND ELECTROSTATICS

AN INNOVATIVE APPROACH
TO WATER AND WASTE TREATMENT

OBJECTIVE:

To elucidate the effectiveness of this instrument in various industrial applications and its possible water reuse potential.

EXPERIMENTAL:

The effects of the Aqua Scrubber on equipment used by the Soft Drink Bottling Industry were measured, also, the effects on cooling towers, boilers, and wastewater treatment systems.

CONCLUSIONS:

This instrument has a definite water reuse application in the Soft Drink Bottling Industry, for cooling towers, boilers, and potential for water reuse in tertiary treatment.

PROCESS:

AQUA SCRUBBER III PHASE WATER
MANAGEMENT SYSTEM

OPEN-CHANNEL ULTRAFILTRATION SPIRAL MODULE FOR WATER REUSE

J. A. Pisani and F. M. Velterop
Envirogenics Systems Company

INTRODUCTION

Ultrafiltration is a pressure driven, membrane separation process which is based on the molecular weight and size of the dispersed or dissolved constituent to be separated. Membranes are available which are capable of retaining suspended material and dissolved species, having high molecular weight, such as colloids, protein, carbohydrates and synthetic resins. These membranes do not retain low molecular weight solutes such as salts, sugar or low molecular weight organic material.

The driving pressure for this membrane separation process is generally low, in the range of 50 to 100 psi, rarely as high as 150 psi. The permeation rate reaches a maximum level, depending upon the membrane porosity and molecular weight of the constituents and pressure applied, however this pressure plateaus at a given level for each process above which further increases in pressure have little increase on the performance of the process.

Low pressure driving force is possible due to the small differences of the osmotic pressure between the feed liquid and the liquid permeate. Consequently the permeate rates for ultrafiltration are more affected by the flow pattern than high pressures which may cause concentration polarization, fouling and plugging of the membrane pores.

Geometric Considerations for Ultrafiltration Processes

Basically two types of ultrafiltration modules and membranes have been developed for water reuse applications. However, it has become evident that the limiting factor in improving economics and productivity has been the membrane system geometry and the resulting flow dynamics. Two geometries which have been utilized to date are tubular and spiral wound membrane systems.

Spiral wound membrane systems provide an economically feasible process; however, they suffer from an irreversible particulate collection within the membrane element. This deleterious affect is observed in applications such as paper mill effluent and other similar industrial waters, containing high suspendid solids which reduce the flux significantly causing plugging of the membrane and flow maldistribution. This flux maldistribution occurs both within the element and between elements in parallel configuration which further aggravates the membrane particulate plugging.

Tubular membrane elements, on the other hand, function very satisfactorily in the presence of high suspended solids. Nevertheless due to the fact that these systems require high recirculation flows of feed solution, to achieve high product water recoveries and due to their low membrane area packing density, cost approximately twice as much as a spiral wound membrane system initially. Cost of operation and maintenance is high due to the high energy requirements and pressure drop.

In order to combine the advantages of each of the ultrafiltration concepts; namely, spiral wound and tubular membrane system, an open channel spiral wound element has been developed. The geometry of the open channel allows almost the same membrane packing density as the spiral wound, while providing the flow through characteristics similar to the tubular membrane system. By producing the open channel configuration without a spacing material between the membrane surfaces, maldistribution of flow and plugging is reduced or eliminated. The resulting open channel membrane element exhibits a high recovery, high membrane packing density, low cost pressure containment and low pressure drop.

Dynamic Flow Pattern of the Open Channel Element

Feed water flows into the open channel element through the center tube. The center tube is slotted to permit flow into the open channel and contains a plug at a point in the center tube which corresponds to the ratio of feed to concentration, thus fixing recovery rate for the element. The feed water passes through the slits, into the open channel made up of the active membrane surfaces; it is directed across the surface of the membrane with permeate flowing through the membrane and concentrate is moved through slits in the center tube. Figure 1 depicts the configuration of the open channel membrane elements. The permeate (flux or product water) exits the module via transport material through the unsealed edge.

In a conventional spiral wound element the product water must traverse a longer distance from the end of the leaf to the center product tube, especially in large diameter elements, causing considerable pressure drop. Whereas in the open channel configuration, the flow path for the product water is at most one-half the length of the element. Resulting in an estimated benefit of 10 to 40% less pressure drop, depending upon the nature of the feed water and the process conditions. Additional benefits are derived from the fact that the feed water (or solution) is unrestricted, which eliminates particulate plugging and maldistribution in the flow pattern.

Due to the fact that you do not use spacer material, between the leafs, pressure drop in the feed channel is lower than conventional spiral wound and the length of the leaf is restricted only by the pressure drop and desired recovery.

In order to reduce the effect of pressure drop and flow velocities, a choice of the ratio of feed slots to reject slots can be adjusted to accommodate various recoveries desired. As a consequent with one leaf length, one may vary the recovery to any predetermine ratio desired, while maintaining a constant flow velocity across the membrane surface.

Types of Membranes Use in the Open Channel Element

The membrane which exhibits the greatest versatility in the construction of the open channel element is the polysulfon membrane is cast on a woven support cloth; exhibiting anisotropic characteristics i.e., it consists of a very thin, microporous skin, with a relatively thick substructure which is highly porous. The polymer is hydrophobic in nature. Membrane which are based on polysulfon polymer are also very stable, have high resistance to hydrolysis and oxidation. In addition to these stability characteristics of the membrane, it exhibits excellent temperature stability.

Therefore, all applications which require stability to high temperature, hydrolysis, and oxidation resistance such as commercial laundries, food processing application are excellent candidates for these open channel elements utilizing the polysulfone membrane for water reuse.

It is particularly noteworthy to emphasize the ability of this membrane to withstand very aggressive oxidative cleansing which makes it an excellent candidate for the food processing industry.

Table 1 is a summary of the characterization of the polysulfon based membrane.

Alternatively for applications in which the feed solutions contain hydrophobic impurities such as oil-water emulsions, the polysulfon membrane is not a practical choice. A membrane consisting of polyacrylonitril is available for these applications. The polyacrylonitril membrane is also more compatible with feed solutions which exhibits low pH characteristics.

TABLE 1

Summary of Polysulfone Membrane Properties & Characteristics

Initial Clean Water Productivity	150-200 gfd at ~ 60 psi (4 bar) about 25°C
Productivity in Water Reuse Applications	Dependent upon the nature of the feed solution with regard to S. S. solid, dissolved solids, etc.

Rejection Properties

Solute	Rejection %
Human Albumin (mol wt. 65,000)	100
Polyoxyethyleenglycol (mol wt. 20,000)	95
Lactalbumin (mol wt. 16,000)	100
Inulin (mol wt. 5000)	5
Inorganic Salts	0

Temperature Resistance

Operation	T°C
Acceptable Temperatures for Continuous Operation	60 to 70
Short Terms or Poure (e.g. Cleaning Operation)	90

pH Resistance

Operation	pH
Acceptable for Continuous Operation	3 to 13
Short Term Exposure (e.g. Cleaning Operation)	14

TABLE 1 (continued)

Oxidation Resistance

Operation	Cl_2ppm
Acceptable for Continuous Operation	~ 50
Short Term (e.g. Cleaning Operation)	2000-3000

Membrane Testing Procedures

After the membrane has been cast, its properties are determined using a standard round laboratory membrane test cell, in which a $28cm^2$ section of active membrane is placed. The flow velocity through the cell and along the membrane surface varies from 40 to 120 cm/sec and is operated under a pressure of 60 psi (4 bar). The flux (or product flow rate) of the membrane is measured immediately upon starting and after 60 minutes of testing. At the 60 minute test 0.1% of human albumin is introduced to determine the rejection rate.

Open Channel Element Testing

Spiral wound open channel elements have been constructed in 4-inch diameter by 30-inch long elements. These elements have been tested up to pressures of 100 psi (6.8 bar) in the apparatus depicted in Figure 2. As is evident from the figure, the element testing is performed by connecting feed water to the center tube on the feed side collecting product in the trough and concentrate from the center tube.

Since this is an on-going program, additional testing will be conducting introductory contaminated feed water from various industrial sources to illustrate the reuse possibilities.

Typical Productivity Calculations for a Single Open Channel Element

In Table 1 we have reported the clean water productivity for the membrane as 150-200 gfd about 60 psi, (4 bar) and 25°C. One might assume, for purpose of illustrating the capacity in real terms, under actual operation with a water which contains some suspended solids and other dissolved contaminates that a flux (productivity) of 20 gfd is achievable. Further, if we factor into our calculations a pressure drop of 20 psi (1.4 bar) across a single element and if we compute the area of membrane available for an element 12 inches in diameter and 36 inches in length, the area of membrane would be 750 sq ft ($70M^2$), then the estimated product flow per day will be 15,000 gallons per day ($56.8M^3$). Clearly the advantage of this open channel configuration is evident.

Typical Applications

From preliminary testing (no complete module testing has been conducted for periods of more than one hour with industrial water) it appears the open channel ultrafiltration module will find application for water reuse, in the following industries:

Food Process
Commercial Laundry
Textile
Metal Working
Paper & Pulp

Long term industrial pilot testing will be required to complete actual field data to establish the efficiency of the open channel membrane module.

Conclusions

A viable spiral wound open channel ultrafiltration membrane element has been developed for water reuse applications.

envirogenics systems co.	Open-Channel UF Module	DRAWN J. CALVELLI 2-77	PROJECT	DRAWING NO.	REV
		CHECK	ENVIROGENICS	SCALE	
		DESIGN	CUSTOMER	REL DATE	

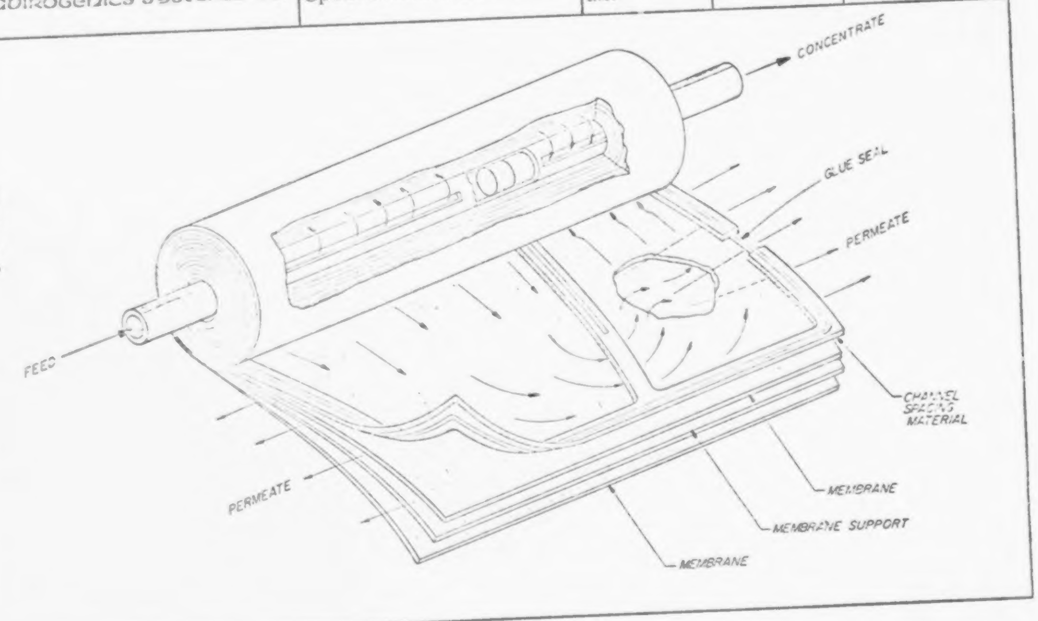


Figure 1

envirogenics systems co. EL MONTE, CALIF.	UF MODULE TEST APPARATUS	DRAWN VB 4-3-1979 CHECK APPROVE	DRAWING NO. 2-2056 SCALE NONE REL DATE	REV -

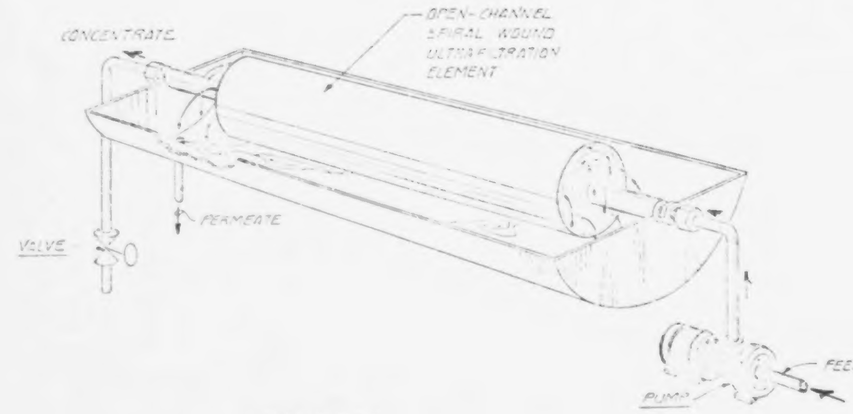
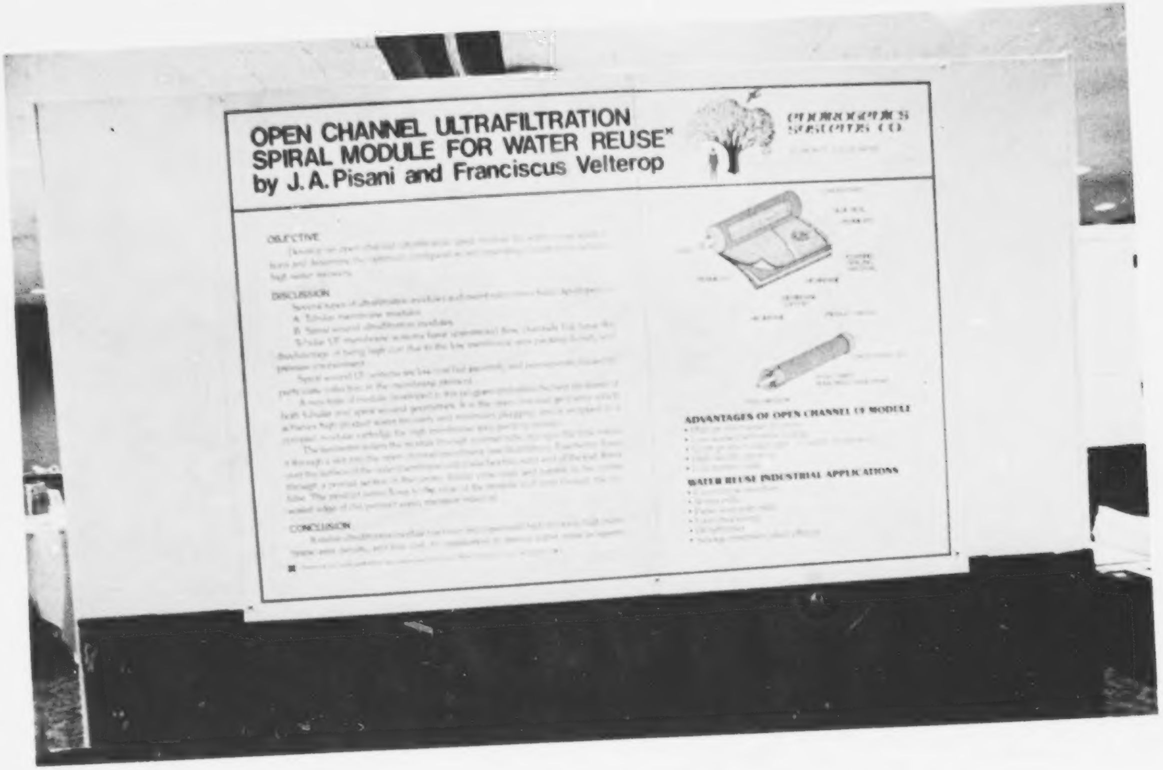


FIGURE 2



OPEN CHANNEL ULTRAFILTRATION SPIRAL MODULE FOR WATER REUSE™
by J. A. Pisani and Franciscus Velterop

OBJECTIVE

DISCUSSION

CONCLUSION

ADVANTAGES OF OPEN CHANNEL UF MODULE

WATER REUSE INDUSTRIAL APPLICATIONS

QUATERNARY AMMONIUM RESINS AS CONTACT WATER DISINFECTANTS

Gilbert E. Janauer
Associate Professor of Chemistry
State University of New York at Binghamton, NY 13901

Ilona H. Walfish
Chemistry Graduate Candidate
State University of New York at Binghamton, NY 13901

Marvin Gunz, Madeline Lalia, and Susan Marcus
Independent Research Students
State University of New York at Binghamton, NY 13901

Chlorination is without any doubt the most reliable water disinfection method and at this time the only acceptable one for large scale use in the United States, although a potential health risk is posed by traces of chlorinated hydrocarbons and similar compounds[1,2,3] formed by reactions with organics present in water during treatment with chlorine. The last few years have seen a surge in interest and research activity with the objective of finding effective alternatives to chlorination, viz. disinfection processes which would produce and introduce lesser amounts or none of these toxic or carcinogenic substances into community water supplies. Most of the proposed alternatives would rely on water treatment with some other soluble oxidizing agent including ozone[4,5,6,7,8,9] or on irradiation with ultraviolet light.[9,10] Also suggested were combinations of several different processes and the elimination of the practice of prechlorination[9] based on the belief that prechlorination accounts for producing the major fraction of organohalo compounds, although it was pointed out that heavy-dose prechlorination is required only when raw water supplies are grossly polluted.[11] One novel approach to water disinfection would make use of the oxidising power of polyhalide species held (sorbed) strongly on columns of commercial strong base anion exchange resins.[12,13,14] In that case the polyhalide counterions on the resin exchange, react, and release free halogens and/or hypohalous acids, apparently at the rate required for the destruction of harmful organisms contained in the water being passed through the column. It so happens that the most effective among the polyhalide species contain iodine (e.g., in form of the triiodide ion, I^{3-}) which might prohibit general application in view of possible restrictions on dietary intake of iodine for certain groups in the population.

A completely different class of compounds was recently suggested for possible use in drinking water disinfection,[15] namely the well-known soluble quaternary ammonium salts (Quats). Quats had been suggested before as a possible means for emergency drinking water disinfection[16] and have been widely used, particularly as sanitizers, in commercial applications on account of their very low toxicity for mammals and their wide spectrum of antimicrobial activity.[17,18] Although Quats had been misused earlier and had, therefore, fallen into disrepute at least among medical practitioners, they are recognized as powerful biocides and the increased antimicrobial efficacy of a "second generation" of soluble N,N-dimethylbenzylalkyl Quats was recently mentioned.[18]

Based on the assumption that Quats would retain their intrinsic biocidal properties when incorporated into a suitable polymer matrix a new approach was taken in this laboratory.[19] Among preparations tested in preliminary experiments only a N,N-dimethyldodecylbenzylpolystyrene resin (Resin 12) exhibited strong contact bactericidal action against *E. coli*, a gram-negative bacterium. These preliminary results were subsequently substantiated[20] and a systematic investigation initiated. Results presented show indeed that Resin 12 acts as a powerful contact bactericide against a gram-positive bacillus as predicted from the intrinsic properties of the soluble Quat analog.

MATERIALS AND METHODS

Bacteria and Cultivation

Bacillus subtilis (NP-40) was used in all experiments. Continuous broth cultures of *B. subtilis* were grown at 40°C for 14-16 hrs in Buffered Glucose Broth.[21] The transfer inoculations were made daily according to the standard procedure described in "Laboratory Methods in Microbiology."[22]

Resin Beds

Commercial chloromethylated crosslinked polystyrene (200-400 mesh, 2% DVB) was aminated with N,N-dimethyldodecyl amine and quaternized using a procedure resembling conventional anion exchange resin synthesis as described elsewhere.[23] The resulting resin was cycled three times alternating with 50 ml 2 M HCl and 50 ml 0.1 M NaOH, then treated several times with 25 ml portions of 2 M NaCl, and finally washed with deionized, distilled water (DDW) until the effluent was free of chloride. The resin was stored in DDW. Using the Volhard procedure for exchangeable chloride[24] or an acidimetric procedure with resin in the hydroxide form, Resin 12 ion exchange capacity was determined as 0.45 ± .02 meq/ml of resin bed for different batches synthesized.

All resin beds used (Resin 12, Dowex 1X2, and the parent copolymer, i.e., chloromethylated polystyrene) were packed as follows: the meniscus of 1.0 ml DDW, in a 6 cm x 0.8 cm^2 fritted, glass column was marked with a line. Resin was added as a slurry in DDW so as to settle into a bed to the height of this mark. Resin 12 was used in the chloride form in all experiments except in those for testing the effect of gegenions.

Preparation of *B. subtilis* Test Solutions

For all experiments, cells grown for 14 hrs (in buffered Glucose Broth at 40°C) were centrifuged at 24°C, 5000 rpm for 7 min in a Sorval Superspeed RC-2 Automatic Refrigerated Centrifuge. The pellet was suspended in 250 ml of sterile 0.01 M Tris/HCl buffer (pH 7.6) and test solutions were made by diluting 10 ml of this suspension with more buffer, up to 2000 ml. Portions of all such test solutions to be used in each experiment were combined, aseptically, to form a Control solution (approximately 2 x 10^5 cells/ml at start). In each and every test run, a Control solution was kept under completely identical conditions to the test solutions.

Test of Bactericidal Action

Aliquots of test solutions (containing equal amounts of bacteria) were passed through each resin bed from sterile 1000 ml separatory funnels attached to the column tops by short pieces of Tygon tubing at flow rates ranging from 10 to 12 ml/min. Effluents were collected in 100 ml fractions at predesignated intervals in sterile, enclosed beakers and then monitored for cell density as follows: 0.1 ml portions of undiluted, as well as diluted, effluent samples were plated, in triplicate, on Nutrient Agar (in 100 x 15 mm petri dishes) by the spreader method[25] and incubated overnight at 40°C. Viable cells were counted, as described in "Standard Methods for the Examination of Water and Wastewater."[26] Typically, the counts on triplicate plates of the same Control sample were within $\pm 0.2 \times 10^5$ cells/ml of each other (for a 1000-fold dilution, the counts were: 19, 20, 21). Results were expressed as # viable cells/ml. Percent viable was obtained as (total # viable bacteria emerging divided by total # bacteria passed through) times 100. The total mass of bacteria passed through each bed was monitored throughout each run by determining the cell density of the Control at the same predesignated intervals.

Procedure for Kill Capacity

Kill capacity was operationally defined as the total number of cells applied to a resin bed until the first appearance (i.e., detection) of viable organisms in an effluent fraction. At room temperature (24°C ± 2°C) test solutions were passed through the standard Resin 12 beds at flow rates from 10-12 ml/min. The 100 ml effluent fractions were collected and monitored at predesignated intervals corresponding to the 200, 400, 800, 1200, 1500, 1800, 1900, 2100 and 2300 ml points. In experiments at elevated temperature (35°C ± 3°C) the beds, all test solutions, and the Control were kept in a temperature-regulated oven for 1 hr prior, and throughout, the runs. The predesignated collection intervals were the 400, 800, 1000, 1200, 1500 and 1800 ml points. All runs at room temperature were carried out in triplicate, those at 35°C in duplicate. Kill capacity was not affected by small changes in flow rate but seemed to vary to a slight extent with concentration (as opposed to total mass) of *B. subtilis* in the test solutions.

Resin Bed Regeneration

Regeneration experiments consisted of three steps, all performed at room temperature. During Step I, "Exhaustion," 4000 ml of test solution containing from 8×10^8 to 2×10^9 of *B. subtilis* were passed through each column and only the last 100 ml effluent fraction was collected and monitored. Exhaustion was arbitrarily defined as addition of cells until a point where $\geq 50\%$ of those applied emerged viable. In Step II, "Regeneration," 250 ml of various regenerant solutions were passed through the treated column, followed by 250 ml buffer (see above). In Step III, "Testing of Regenerant Efficiency," between 1200 and 2300 ml of test solution were passed through each treated Resin 12 bed and effluents were collected and monitored at predesignated intervals as described in the foregoing section.

RESULTS

Kill Capacity

Resin 12 effectively kills *B. subtilis* on short contact, the parent matrix (chloromethylated, DVB-crosslinked polystyrene) effects some initial "filtering," and this filtering (or adsorbent) action is not significantly enhanced by intermittent rinsing of beds with 2 M HCl. Figure 1 shows that of 5.8×10^7 *B. subtilis* cells passed through an HCl-treated Resin 12 bed, none emerged viable, while an increasing percentage emerged viable as additional increments of test solution were passed through parent matrix beds. Initially, the parent matrix traps a certain percentage of cells, but this filtering action is small compared to the biocidal action of Resin 12 for a large number of cells.

The bactericidal potency of Resin 12 was clearly demonstrated by its kill capacity in terms of total bacterial mass when a greater total quantity of cells was applied at room temperature. As shown in Table I, no viable cells emerged from the beds until 8.2 x 10^8 of the total 9.4 x 10^8 cells had contacted the resin bed. From this point on, less than 1% of the total bacteria applied emerged viable from the beds. The tremendous kill capacity of Resin 12 (>8 x 10^8 cells) by short contact with a 1 ml bed is most striking in view of the fact that the chloride form of conventional high capacity polystyrenebenzyltrimethylammonium anion exchange resins (Dowex 1 type) exhibits no antimicrobial activity whatever.[12] Neither Dowex 1, nor the n-octyl ("Resin 8"), or the n-octadecyl ("Resin 18") analogs of Resin 12 had shown significant bactericidal activity against *E. coli* during preliminary tests in this laboratory[19] although a small percentage of applied cells always was "filtered" off (sorbed by the resin beds). This was found in the present study with *B. subtilis* as well and is in agreement with the well-known chainlength dependence of the antimicrobial efficacy of soluble aliphatic Quats.[17]

An enhancement of Resin 12 bactericidal action by elevated temperature is also seen from the data given in Table I. No viable cells were detectable in the effluent after 9.6 x 10^8 cells had been applied to the resin bed at 35°C. The enhancement of antimicrobial activity of soluble Quats with increasing temperature in solution is a well established fact[27] and therefore, probably, not caused by any special effect associated with the polymer.

Resin Regeneration

Clean-up studies showed that Resin 12 can be effectively regenerated by washing used beds with EtOH/HCl mixtures. However, kill capacity decreases gradually with repeated cycling (treatment--regeneration). As seen in Fig. 2 regeneration efficiency of the cleaning mixtures increases with higher HCl concentration for a fixed EtOH fraction, and with increasing EtOH concentration for a constant HCl fraction. The EtOH-dependence may be a consequence of the solvent selectivity of ion-exchange resins swollen in H_2O-EtOH mixtures: as the EtOH content of the solvent system approaches near 100%, the pronounced selectivity of the resin phase for the aqueous component in the mixture decreases sharply.[28] Accordingly, a larger amount of the organic solvent will then be imbibed by the resin. The organic solvent would be expected to better solubilize and elute sorbed cellular fragments which may clog active Quat sites on the polymer matrix.

Effect of Gegenion

In a series of duplicate runs a total amount of 2.4 x 10^9 *B. subtilis* cells each was applied in volumes of 1600 ml to standard columns of Resin 12 in various ionic forms. Gegenions (counterions) employed were chloride (as in all other experiments, to provide a reference value), bromide, iodide, thiocyanate (SCN^-), ethanesulfonate ($EtSO_3^-$), and n-pentane sulfonate ($PentSO_3^-$). No viable cells emerged in the case of the halide gegenions, but 33% viable organisms were found in the last 100 ml fraction in the case of SCN^-, 10% viable in the last fraction of the $EtSO_3^-$ resin, and 1% viable in the case of the columns loaded with $PentSO_3^-$ as the gegenion. These results indicate a significant effect of the properties of individual counterions on kill capacity which needs further quantitative study with practical implications for water disinfection in mind.

DISCUSSION

Resin 12 is a powerful contact bactericide by virtue of the intrinsic biocidal action of the N,N-dimethyldodecylbenzylammonium function attached to the polystyrene matrix. These active sites are used up during a disinfection process but can be regenerated with a suitable elutrient. In addition to being a potent bactericide Resin 12 can be expected to be effective against many other small life forms in analogy to soluble, long chain Quats.[17,18] The polystyrene matrix of Resin 12 is stable and the cationic Quat sites are strongly attached. Any displaced counterions (chloride) are innocuous.

In Resin 12 a major fraction of the aromatic nuclei of the polystyrene backbone is functionalized resulting in a fairly high ion exchange capacity despite the large size of the n-dodecyl chain. However, it is likely that only those Quat sites attached in the outermost layer of the polymer beads can interact with micron size organisms which are mechanically excluded from the interior of the gel network. It is suggested that the strong bactericidal activity of Resin 12 is due to the availability of the extremely high concentration of Quat sites on the resin surface--the swollen resin has an overall formal "molarity" of almost 0.8 M. The actual concentration at the bead surface may even exceed this concentration and will exceed by a factor of up to 100 (!) the highest concentration of soluble Quats ever used in solution--a one percent solution is $\sim 3 \times 10^{-2}$M. A second important factor is the expected strong, general hydrophobic interaction driving the bacteria from the structured aqueous phase toward the bead-solution interface. In addition, the negatively charged cells are strongly attracted to the positively charged active sites. Furthermore, a special interface effect may operate at the resin/solution boundary and may increase biocidal potency. There is also a good possibility that modified resin types (now under investigation) may show further enhanced performance. Resin 12 and similar polymers warrant additional research effort since they may provide a viable alternative to chlorination and other water disinfection methods based upon the addition of reactive, soluble chemicals. Even now such resins may be applicable under circumstances where strong residual protection in water after treatment is not of the essence. For example, the application of Resin 12 or similar polymers in small water treatment systems of the U.S. that currently use no disinfection process[29] would seem to hold much promise.

The efficacy of Resin 12 against a variety of microorganisms including actual pathogens must be tested. Some earlier reservations concerning soluble Quats, e.g., that they are active only against lipophilic viruses[30] and are not sporicides[31] may not hold for high-surface concentration interface-active polymeric Quats. Previous observations on possible activity of soluble Quats against protozoa[32] suggest that the polymeric Quat resins may even be effective against such resistant pathogens as amebic cysts.[16] Algicidal and fungicidal properties can be predicted as probably quite strong also in analogy to the properties of soluble Quats.[18,30]

RELEVANCE TO WATER REUSE

There seems to be no doubt that some form of primary chlorination will continue to be the most cost-effective way to ensure that safe, high quality potable water can be furnished to municipalities in the U.S.

and in most other countries. The chlorine residual protection appears to be a must, and even a superior insoluble contact biocide would--by its very insolubility--not impart any protection against secondary pollution from leakage into the distribution system. However, there may exist other situations where Quat resins (or other powerful insoluble biocides) might offer advantages. In principle, disinfection by such resins could be employed at any stage of a potable water treatment process train except at the final stage prior to distribution (in small systems, emergency kits, etc. Quat resins would suffice alone--secondary invasion is improbable) as long as the bulk of organics and suspended solids would be removed by conventional processes which include a filtration step beforehand. This would be even more true for all those advanced systems which include an ion exchange or membrane process. Again the Quat resins could provide the major disinfection function with chlorination reserved for the final, crucial treatment prior to distribution when a minimum of organics would be present to form toxic compounds. There may exist a number of other important potential reuse-related applications. These would also make use of Quat resins in sequence with other adsorbent/filtering devices. Thus, the recycling cooling systems of power plants, boiler feed waters, and swimming pools could be kept free of deleterious microorganisms of all kinds. Also, the noxious growth of microlife forms on activated carbon filters could possibly be eliminated, and swimming pools with recycling could be kept free of pathogens as well as algae. These and other thinkable applications such as the final disinfection of treated waste water before discharge into a river, or treatment of water before injection for the purpose of combatting saline water intrusion in coastal areas may or may not remain of mere academic interest depending on the real life performance of Quat resins and, of course, cost effectiveness.

REFERENCES

1. BELLAR, T.A.; LICHTENBERG, J.J.; & KRON, R.C. The Occurrence of Organohalides in Chlorinated Drinking Water. *Jour. AWWA*, 66:12:703 (Dec. 1974).
2. Preliminary Assessment of Suspected Carcinogens in Drinking Water. Interim Report to Congress. U.S. Environmental Protection Agency. Appendix VII. Health Effects Caused by Exposure to Contaminants. Washington, D.C. (Dec. 1975).
3. DOWTY, B.J.; CARLISLE, D.R.; & LASETER, J.C. New Orleans Drinking Water Sources Tested by Gas Chromatography-Mass Spectrometry. *Environ. Sci. and Tech.*, 9:8:762 (Aug. 1975).
4. WHITE, C.G. Disinfection: Present and Future. *Jour. AWWA*, 66:12:689 (Dec. 1974).
5. AUGENSTEIN, H.W. Use of Chlorine Dioxide to Disinfect Water Supplies. *Jour. AWAA*, 66:12:716 (Dec. 1974).
6. KAFZENELSON, E.; KLETTER, B.; & SHUVAL, H.J. Inactivation Kinetics of Viruses and Bacteria in Water by Use of Ozone. *Jour. AWAA*, 66:12:725 (Dec. 1974).
7. KELLER, J.W.; MORIN, R.A.; & SHAFFER-NORTH, T.J. Ozone Disinfection Pilot-Plant Studies at Laconia, N.H. *Jour. AWAA*, 66:12:730 (Dec. 1974).

8. MCCARTHY, J.J. & SMITH, C.H. A Review of Ozone and Its Application to Domestic Wastewater Treatment. *Jour. AWAA*, 66:12:718 (Dec. 1974).

9. HOEHN, R.C. Comparative Disinfection Methods. *Jour. AWAA*, 68:6:302 (1976).

10. SHECHMEISTER, I.L. Sterilization by Ultraviolet Radiations, *Disinfection, Sterilization and Preservation* (Seymour S. Block, editor). Lea and Febiger, Philadelphia (2nd ed., 1977) pp. 522-541.

11. WHITE, G.C. Notes and Comments (answer to article by HOEHN, R.C., reference 9). *Jour. AWWA*, 68:6:308 (June 1976).

12. TAYLOR, S.L.; FINA, L.R.: & LAMBERT, J.L. New Water Disinfection: An Isoluble Quaternary Ammonium Resin-Triiodide Combination that Releases Bactericide on Demand. *Appl. Microbiol.*, 20:5:720 (Nov. 1970).

13. LAMBERT, J.L. & FINA, L.R. Demand Bactericide for Disinfecting Water and Process of Preparation, U.S. Patent 3,923,665, Dec. 2, 1975.

14. MILLS, J.F. Control of Microorganisms with Polyhalide Resins. U.S. Patent 3,462,363, Aug. 19, 1975.

15. WANG, L.K. & PEERY, G.G. Disinfection with Quaternary Ammonium Compounds. *Water Res. Bull. (AWWA)*, 11:5:919 (Oct. 1975).

16. KESSEL, J.F. & MOORE, F.J. Emergency Sterilization of Drinking Water with Heteropolar Cationic Antiseptics. I. Effects Against Cysts of Entamoeba Histolytica. *Am. Jour. Trop. Med. Hyg.*, 26:3:345 (May 1946).

17. PETROCCI, A.N. Quaternary Ammonium Compounds, *Disinfection, Sterilization and Preservation* (Seymour S. Block, editor). Lea and Febiger, Philadelphia (2nd ed., 1977) p. 325.

18. PRYOR, A.K. & BROWN, R.S. Quaternary Ammonium Disinfectants. An Updated Perspective. *Jour. Environ. Health*, 37:4:326 (Jan./Feb. 1975).

19. NUDEL, R. Quaternary Ammonium Resins: An Alternative to the Chlorination of Water? Final Summary Report of Student Summer Project. NSF-SOS 547A. (June 1977). Presented but not published.

20. WALFISH, I.H. Water Disinfection with Quaternary Ammonium Resins. MA Thesis. State University of New York at Binghamton, Binghamton, N.Y. (1979).

21. American Public Health Assoc. *Standard Methods for the Examination of Water and Wastewater*. American Public Health Assoc., Inc., New York (12th ed., 1965) p. 586.

22. HARRIGAN, W.F. & MCCANCE, M.E. *Laboratory Methods in Microbiology*. Academic Press, New York (1966) p. 15.

23. WALFISH, I.H. & JANAUER, G.E. unpublished.

24. KOLTHOFF, I.M.; SANDELL, E.B.; MEEHAN, E.J.; & BRUCKENSTEIN, S. *Quantitative Chemical Analysis*. The Macmillan Company, New York (4th ed., 1969) p. 798.

25. LYNE, C. *Microbiological Methods*. University Park Press, Baltimore (1940) p. 155.

26. American Public Health Assoc. *Standard Methods for the Examination of Water and Wastewater*. American Public Health Assoc., Inc., New York (12th ed., 1965) p. 593.

27. QUISNO, R. & FOTER, M.J. Cetylpyridinium Chloride. I. Germicidal Properties. *Jour. Bacteriol.*, 52:1:111 (Jan. 1946).

28. GREGOR, H.P.; NOBLE, D.; & GOTTLIEB, M.H. Studies on Ion-Exchange Resins. XII. Swelling in Mixed Solvents. *Jour. Phys. Chem.*, 59:1:10 (Jan. 1955).

29. State of the Art of Small Water Treatment Systems. U.S. Environmental Protection Agency. Office of Water Supply, Washington, D.C. (Aug. 1977).

30. LAWRENCE, C.A. *Surface Active Quaternary Ammonium Germicides*. Academic Press, New York (1950).

31. HEDGECOCK, L.W. *Antimicrobial Agents*, Vol. 3, Medical Technology (Ruth M. French, editor). Lea and Febiger, Philadelphia (1967) p. 71.

32. FAIR, G.M.; CHANG, S.L.; TAYLOR, M.P.; & WINEMAN, M.A. Destruction of Waterborne Cysts of Entamoeba Histolytica by Synthetic Detergents. *Am. Jour. Public Health*, 35:2:228 (Feb. 1945).

TABLE I

Kill Capacity of Resin 12

Temperature	Resin Batch	Amount Cells Applied	Kill Capacity*	% Viable of Total
24°C	1 a	9.4×10^8/2300 ml	8.2×10^8	<1
	b	same	same	same
35°C	1 a	9.6×10^8/1800 ml	≧total	0
	b	same	same	same
24°C	2 a	9.4×10^8/2300 ml	≧total	<1
	b	same	same	same
24°C	3 a	2.4×10^9/1600 ml	≧total	0
	b	same	same	same

*Operationally defined as the total number of cells applied to first appearance of viable cells in column effluent.

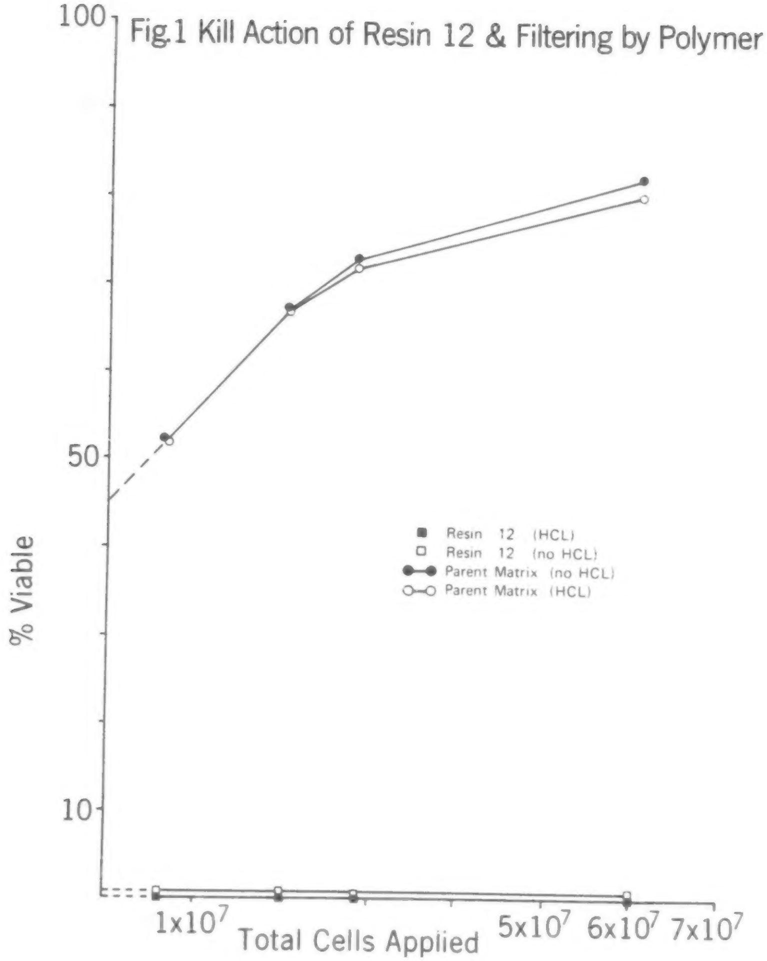
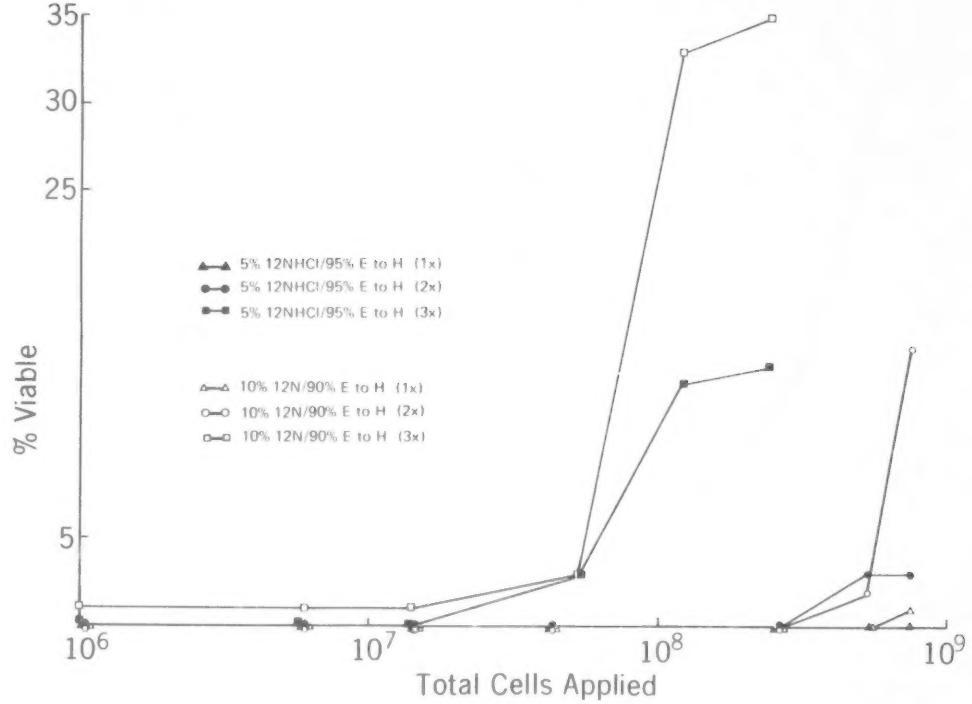


Fig.1 Kill Action of Resin 12 & Filtering by Polymer

Fig. 2 Effect of Repeated Recycling on KC



TAHOE-TRUCKEE WATER RECLAMATION PLANT
FIRST YEAR IN REVIEW

Stanley A. Smith
Department Manager-Wastewater Treatment-CH2M HILL
Robert L. Chapman
Division Manager-CH2M HILL
Ossian R. Butterfield
Manager/Chief Engineer
Tahoe-Truckee Sanitation Agency

INTRODUCTION

The Tahoe-Truckee Sanitation Agency was formed on 1 May 1972 to carry out the mandate of the Porter Cologne Water Quality Act, a California law requiring exportation of all sewage from the Lake Tahoe Basin. Upon its formation, the Agency immediately embarked on a program to plan, program, design, and construct a regional system which would transport all sewage from an area encompassing the California north shore of Lake Tahoe and Truckee to a regional plant. Construction of this new facility would result in the replacement of the existing interim treatment facilities operated by the Agency's five member entities: the Tahoe City Public Utility District, the North Tahoe Public Utility District, the Alpine Springs County Water District, the Squaw Valley County Water District, and the Truckee Sanitary District. The Tahoe-Truckee Sanitation Agency (T-TSA) is governed by a Board of Directors comprised of five appointed directors, one from each member district.

Since it was mandatory that effluent be disposed of outside the Lake Tahoe Basin and in a manner that would protect the quality of receiving waters and potable water sources, the following alternative plans were identified by T-TSA as being worthy of study:

1) Exportation of treated wastewater to Long Valley, Sierraville, or Loyalton for use as irrigation water on farmland--These valleys lie in another basin approximately 30 miles (48 km) north of Truckee.

2) Discharge of highly treated wastewaters to the Truckee River, either in Martis Valley or after the transportation of either treated or untreated wastewater to Reno/Sparks treatment plant located approximately 35 miles (56 km) downstream in Nevada

3) Land disposal of treated wastewater in the Truckee or Carson River Basins

4) Upgrading existing treatment facilities in each of the member districts

Retention of the effluent in the Truckee River Basin was decided to be of prime importance in order that the quantity of water available to downstream users not be diminished by this project. It was also determined that treatment of the wastewater to a high degree at a location near Truckee was the most economical solution and that this plan had the least adverse impact on the environment. Third, the existing high water quality standards required of the Truckee River could not be adhered to by upgrading of the existing treatment plants. The State of California's rigid waste discharge requirements necessitated the implementation of the most sophisticated treatment available.

The project finally selected as the most advisable required the construction of an interceptor line from Tahoe City to Truckee, California, the construction of a 4.83-mgd (212 l/s) regional sewage treatment plant in Martis Valley, and the installation of an underground disposal system that would allow the effluent to percolate into the permeable glacial outwash soil near the plant. Full tertiary treatment, including maximum removals of nitrogen and phosphorus, was deemed necessary to protect the quality of the Truckee River into which the effluent would ultimately find its way and to ensure the safety and integrity of this primary water source for Reno, Nevada, a major city downstream.

Design of the water reclamation plant was initiated in January 1975. The contract for construction was awarded in November 1975 to Del E. Webb Corporation and University Mechanical Contractors, a joint venture. Because of the urgent need for the facility, the contractor was allowed only 24 months to have the complex plant operational and was permitted additional time for final completion. The plant began treating sewage on 1 February 1978, 27 months after award of contract.

TREATMENT REQUIREMENTS AND PROCESSES

The discharge requirements for the plant are very stringent and were established to protect the beneficial uses of the Truckee River and ground-water sources. The discharge requirements limit constituent concentrations in the plant effluent as well as limit constituent increases in the Truckee River. The plant effluent limitations are listed in Table I.

TABLE I

Effluent Limitations

Constituent	Units	Mean Concentration	Maximum Concentration
COD	mg/l	15	40
Suspended Solids	mg/l	2	4
Turbidity	NTU	2	8
Total Nitrogen	mg/l	2	4
Total Phosphorus	mg/l	0.15	0.4
MBAS	mg/l	0.15	0.4
TDS	mg/l	440	-
Chloride	mg/l	110	-
Total Coliform Organisms	MPN/100 ml	-	23

Processes installed to meet the discharge requirements include the following:

- Primary treatment
- Pure oxygen activated sludge
- Lime treatment with two-stage recarbonation
- Dual media filtration

- Activated carbon adsorption with onsite regeneration
- Selective ion exchange using clinoptilolite for ammonia removal

Biological sludges are anaerobically digested. Both biological and chemical sludges are dewatered in a plate and frame filter press and landfilled.

The nitrogen removal process is a unique feature of the plant. The clinoptilolite ion exchange beds are regenerated using a 3-percent salt solution in conjunction with an ammonia removal and recovery process (ARRP). The salt regenerant solution is circulated through an exhausted-ion exchange bed to purge the bed of ammonia. The portion of the salt solution with the highest ammonia concentration, approximately 300 mg/l, is elevated to pH 11.1 using sodium hydroxide. This high pH solution is then clarified and pumped to an ARRP module. An ARRP module is a closed air system consisting of a stripping tower, absorber tower, interconnecting duct work and fan.

Ammonia is stripped from the spent regenerant solution in the stripping tower and absorbed in the absorber tower where the pH is maintained at 2 by addition of sulfuric acid. The end result is that ammonia is removed from the regenerant solution and recovered as concentrated ammonium sulfate. The salt regenerant solution is used repeatedly for regeneration of ion exchange beds and all recovered ammonium sulfate is used as a commercial fertilizer in the Sacramento Valley of California.

Ion exchange used in conjunction with the ARRP is believed to be the best process available to provide the ammonia removal required at this location. The low winter temperatures prohibit effective use of biological nitrogen removal or conventional air stripping.

Another rather unusual and interesting feature of the T-TSA water reclamation plant is that even though the plant effluent is of very high quality, with virtually all contaminants removed, current regulations still prohibit it from being directly discharged into the receiving waters of the Truckee River Basin. This requirement came about because of the pristine quality of the Truckee River in this vicinity and because it is the primary source of the Reno, Nevada water supply. Consequently, as a further precautionary measure, the effluent is discharged into an underground disposal system near the plant.

The combination of advanced waste treatment processes and the underground disposal system is believed to provide the best assurance possible of the protection and preservation of this environmentally sensitive and pristine receiving water.

PLANT PERFORMANCE

Raw Sewage Characteristics

The treatment plant receives wastewaters strictly from domestic and commercial sources; there is no industrial contribution. The wastewater characteristics are fairly typical of a moderate-strength domestic sewage except during spring snowmelt conditions when infiltration and inflow dilute the wastewater about 50 percent. The majority of the year, BOD concentrations are typically in the range of 175 to 250 mg/l and total suspended solids concentrations vary from 175 to 225 mg/l. Total phosphorus and total nitrogen concentrations average about 10 mg/l and 40 mg/l, respectively.

Sewage flows and hence total pollutant loadings are highly variable because of the recreational nature of the area. Maximum loadings historically occur during August which is the peak summer vacation period at Lake Tahoe. Other peak loadings occur during the winter ski season, particularly during the Christmas holidays, Washington's birthday weekend, and Easter week. Peak flows to the plant occur during spring snowmelt conditions concurrent with a holiday period.

Maximum hydraulic loads occurred on the plant during Easter week when daily sewage flows averaged about 4.8 mgd (210 l/s). However, the wastewater was fairly dilute at that time due to the effects of infiltration and inflow. Maximum pollutant loading conditions occurred during the month of August when the plant was operating at about 80 percent of its design loading.

Removals Obtained

The plant began operation on 1 February 1978 when about 0.5 mgd (20 l/s) of sewage was diverted from the Truckee Sanitary District system. During the month of February, construction was still continuing and a good deal of performance testing had not yet been accomplished. By 1 March, the plant was accepting flow from all scheduled sources.

Initial performance of the plant was extremely successful, particularly considering that construction of the plant was not truly complete and the startup occurred during severe winter conditions. The overall performance of the plant in 1978 is best depicted by a series of trend charts. Figures 1-4 are trend charts for the most critical wastewater characteristics, COD, suspended solids, phosphorus, and nitrogen. As a result of infiltration and inflow during the spring snowmelt conditions, the concentrations of each of these parameters was relatively low during the spring. The concentrations jumped dramatically in June and remained high throughout the remainder of the year. Maximum strengths occurred during July and August as was expected.

During the initial months of operation, there was naturally a period of troubleshooting and initial process optimization efforts. While effluent quality was relatively good from the start, the quality in general gradually improved during the initial months of operation. Fairly stable operation was achieved during the last 7 months of 1978, and an exceptional effluent quality was consistently achieved.

Table II summarizes the plant effluent quality during the last 7 months of 1978 with respect to the various constituents included in the discharge requirements. During that time, the plant performance essentially met all of the very stringent discharge requirements with exception of the effluent limitations on total dissolved solids (TDS) and chloride concentration. The effluent limitations on TDS and chlorides were established after initiation of the plant design. Consequently, there are no processes included in the plant design to remove these dissolved inorganic constituents.

TABLE II

Plant Performance June Through December 1978

Constituent	Units	Average Effluent Concentration	Effluent Limitations
COD	mg/l	10	15
Suspended Solids	mg/l	0.8	2
Turbidity	NTU	0.6	2.0
Total Nitrogen	mg/l	2.3	2.0
Total Phosphorus	mg/l	0.15	0.15
MBAS	mg/l	0.04	0.15
Total Dissolved Solids	mg/l	633	440
Chloride	mg/l	206	110
Total Coliform Organisms	MPN/100 ml	<2	23

The TDS and chloride concentrations in the raw sewage, coupled with increases through the plant inherent with the chemical treatment and sludge handling processes, essentially preclude the plant's ability to control the concentrations of these constituents in the effluent. In recognition of these factors, T-TSA staff has requested and hopes to receive a relaxation in the discharge requirements for TDS and chloride. Further optimization of chemical treatment and the ammonia removal system may result in eventual lower levels of TDS and chlorides in the effluent.

Sampling of the Truckee River both upstream and downstream of the plant has indicated the project is meeting its primary objective: returning reclaimed water to the basin with no detrimental effect on water quality. Table III illustrates the high quality of the Truckee River and indicates the only parameters that show significant increase downstream of the plant are TDS and chlorides. It is not known how much of this increase is due to the plant discharge and how much is due to other sources of runoff.

TABLE III

Truckee River Water Quality June Through December 1978

Parameter	Units	Upstream of T-TSA	Downstream of T-TSA
Nitrates	mg/l	0.09	0.09
Kjeldahl Nitrogen	mg/l	0.26	0.28
Total Phosphorus	mg/l	0.04	0.06
Dissolved Organic Carbon	mg/l	1.1	1.1
MBAS	mg/l	0.01	0.01
pH	Unit	7.7	7.6
TDS	mg/l	56	64
Chlorides	mg/l	2.7	6.3

COSTS

The total capital cost of the plant was approximately $19,976,000. Table IV shows this cost distributed according to major process units. Approximately 62 percent of the capital cost of the facility is attributable to the AWT processes. The costs of the sludge handling facilities are included in the costs listed for primary and secondary treatment and chemical treatment.

TABLE IV

Capital Costs

Process Unit	Capital Costs	% of Total
Primary and Secondary Treatment	$ 7,666,000	38
Chemical Treatment and Filtration	4,881,000	24
Carbon Adsorption	3,928,000	20
Ammonia Removal	3,501,000	18
Totals	$19,976,000	100

A major factor that significantly influenced the capital cost is the plant's location. The Lake Tahoe area is historically one of the most expensive areas in California for construction projects. Also, because of the high Sierra location, the facility was designed for snow loads to 150 pounds per square foot and temperatures to minus 45°F (-43°C). The severe climate required that nearly all facilities be covered or enclosed for proper operation and maintenance.

Labor, chemicals, and power are the major costs associated with operation of the treatment processes. Other costs include administration, miscellaneous expenses, and final sludge disposal. These costs are presented in Table V for the period June-September 1978.

TABLE V

O&M Costs by Expense Category

Expense Type	$/Million Gallons	Percent of Total
Labor	580	34
Chemicals	475	28
Power	160	10
Administration	140	8
Final Sludge Disposal	51	3
All Other Expenses	294	17
Totals	1,700	100

In Table VI the operation and maintenance costs are summarized according to major process unit. Approximately 81 percent of the current operation and maintenance costs are associated with the AWT processes. It is anticipated that the cost per million gallons will decrease as plant flows increase and optimization of the chemical dosages continue. (During the period June-September, the raw sewage flow averaged approximately 3 mgd (130 l/s).)

TABLE VI

O&M Costs by Major Process Unit

Process	Cost $/Million Gallons	Percent of Total
Primary and Secondary Treatment	331	19
Chemical Treatment and Filtration	760	45
Carbon Adsorption	125	7
Ammonia Removal and Recovery	484	29
Totals	1,700	100

The cost information presented here was the first effort in analyzing actual plant operations and maintenance costs. It is anticipated that there will be some future shift in the distribution of costs shown in Table VI. The labor costs associated with operating the chemical treatment and ammonia removal systems should decrease as operating procedures become more efficient and remaining equipment problems are resolved. Chemical usage should also decrease with these two processes as optimum dosages are determined. The cost of carbon adsorption should increase due to more frequent regeneration of the carbon. During this first year of operation, the need for regeneration has been held to a minimum because of the high quality virgin carbon.

The cost of operation and maintenance is directly related to the stringent discharge requirements. As an example, the effluent limitation of 0.15 mg/l phosphorus requires a high lime dosage and nearly constant operator attention. Malfunctions must be quickly identified and corrected or the effluent limitation will be exceeded for that day. The same operator vigilance is required for all processes because of the extremely low levels of COD, nitrogen, and solids permitted in the effluent.

CONCLUSIONS

The $20 million T-TSA Water Reclamation Plant was successfully put on line beginning 1 February 1978, 3 years after authorization to proceed with design. Performance data collected during 1978 clearly demonstrate the plant's ability to produce an effluent of superior quality basically in conformance with all of the very stringent effluent limitations, exclusive of chlorides and total dissolved solids. Monitoring of the Truckee River has also demonstrated the project is achieving its primary goal of returning reclaimed water to the basin while maintaining exceptional water quality.

Operation and maintenance costs during the first year of operation are estimated at $1,700 per million gallons. The current sewer service charge is approximately $10 per single family residential unit. While these costs are substantial, they are judged reasonable by most people considering the unique environmental setting and the demanding goals.

Major objectives for 1979 include efforts directed at process optimization plus planning for plant expansion. Considerable emphasis is being given to process optimization in an effort to reduce operating costs and/or improve performance. There is still much to be learned about several of the more complex processes.

The plant has limited reserve capacity for additional connections due to the restrictive funding conditions for initial construction. Facility planning and environmental assessment work is already in progress with the objective of expanding the plant as soon as possible. Part of the current effort includes a ground-water and surface-water monitoring program directed at more clearly defining treatment requirements relative to forthcoming revised water quality objectives.

The Tahoe-Truckee Sanitation Agency has resolved many technical, political, environmental, and financial issues in implementing their regional system; many challenges remain. Meanwhile, the basic objectives of the Agency are being realized and the viability and successful performance of the regional water reclamation plant have been conclusively demonstrated during the first year of operation.

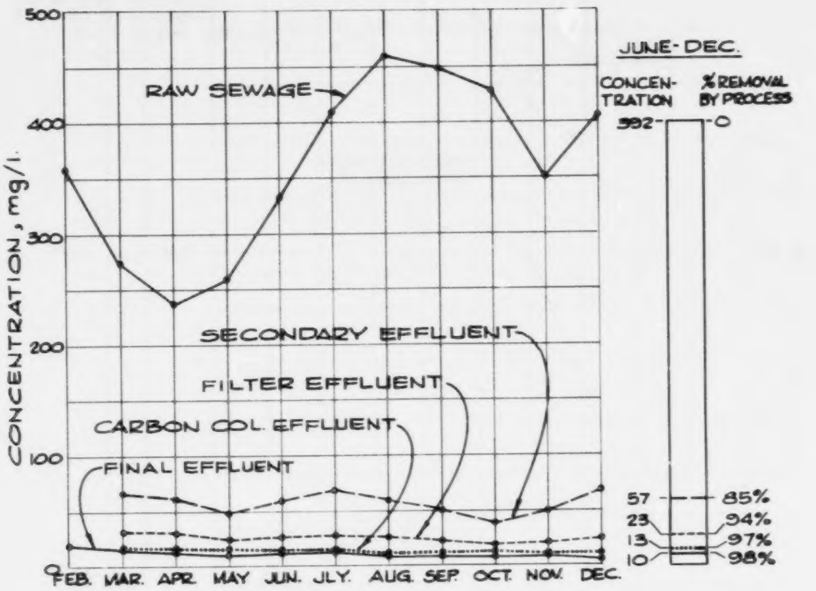


FIGURE 1: COD removal.

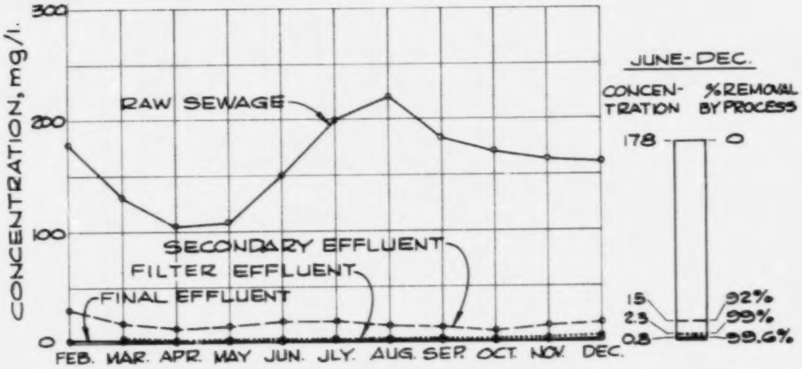


FIGURE 2: Suspended solids removal.

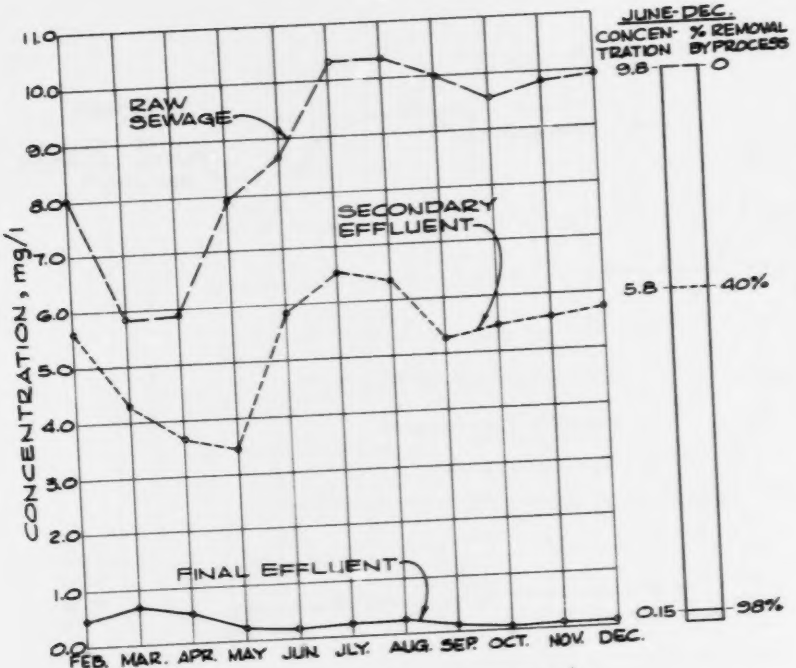


FIGURE 3: Phosphorus removal.

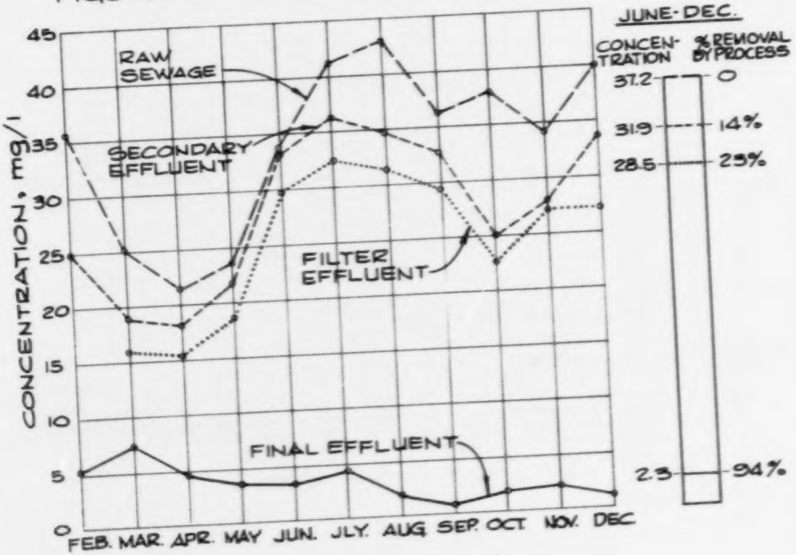
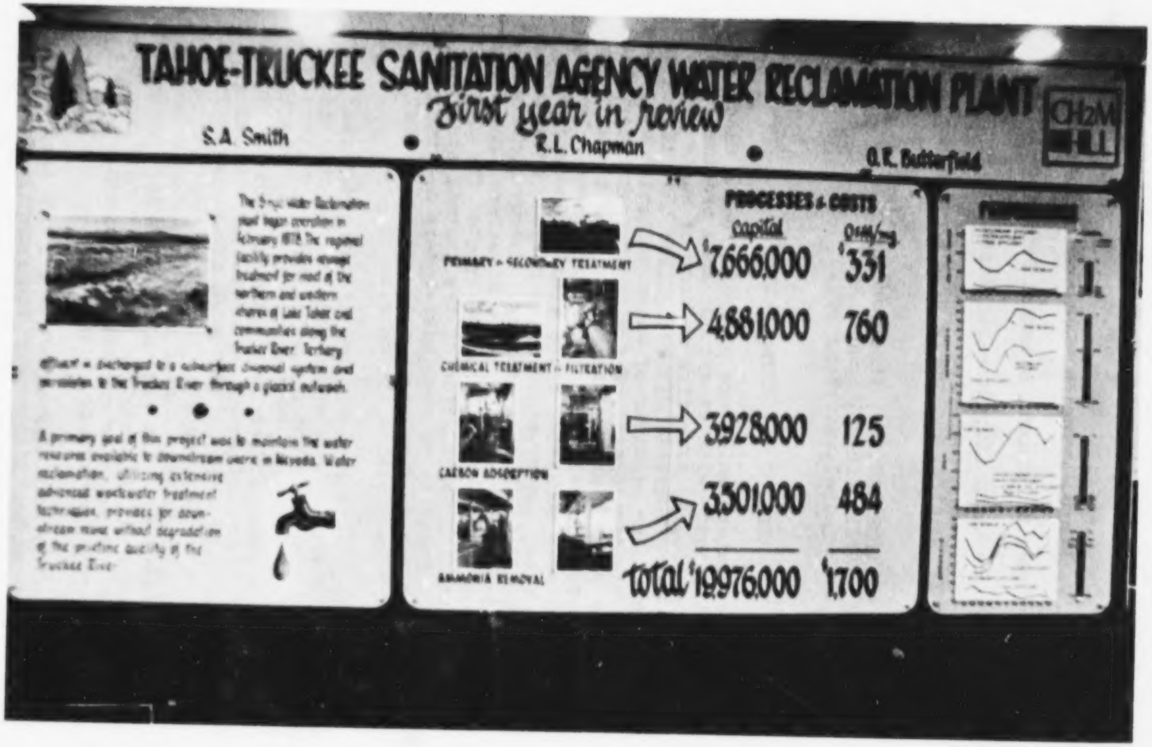


FIGURE 4: Nitrogen removal.



TAHOE-TRUCKEE SANITATION AGENCY WATER RECLAMATION PLANT

First year in review

S.A. Smith • R.L. Chapman • O.K. Butterfield

CH2M HILL

The [illegible] water Reclamation plant began operation in February 1978. The regional facility provides sewage treatment for most of the northern and western shores of Lake Tahoe and communities along the Truckee River. Tertiary effluent is discharged to a subsurface disposal system and percolates to the Truckee River through a glacial outwash.

• • •

A primary goal of this project was to maintain the water resources available to downstream users in Nevada. Water reclamation, utilizing extensive advanced wastewater treatment techniques, provides for downstream reuse without degradation of the pristine quality of the Truckee River.

PROCESSES & COSTS

	Capital	O+M/mg
PRIMARY + SECONDARY TREATMENT	$7,666,000	$331
CHEMICAL TREATMENT + FILTRATION	4,881,000	760
CARBON ADSORPTION	3,928,000	125
AMMONIA REMOVAL	3,501,000	484
TOTAL	$19,976,000	$1,700

"REGENERATION OF WASTEWATERS FOR REUSE THROUGH HYDROPERM™ MICROFILTRATION"

Dr. John E. Santo, Principal Research Scientist

Dr. James H. Duncan, Research Scientist

Dr. Han-Lieh Liu, Research Scientist

Mr. N. I. Shapira, Hydroperm Program Manager

HYDRONAUTICS, Incorporated
7210 Pindell School Road
Laurel, Maryland 20810

SECTION I - INTRODUCTION

Many wastewaters which are presently being discharged into the environment, with or without some treatment, can be reused by employing effective methods of waste treatment. It is being increasingly recognized that the ultimate solution to the problems of water pollution will have to depend on the development of treatment techniques which achieve water reuse, rather than on the traditional methods of treatment for discharge into the environment. Development of techniques for water reuse also holds important advantages. Water conservation is an obvious advantage in areas where water is in short supply. Energy conservation is also an important result if the wastewaters from industries which utilize hot water can be treated for reuse without any substantial degradation of the thermal state of these wastes. Examples of such wastewaters are the hot wash and rinse waters from commercial and industrial laundries, and from restaurants and motels.

In the treatment of both domestic and industrial wastewaters, the removal of suspended solids (and colloids) is a required unit operation which arises under a variety of circumstances. For example, this unit operation is often essential as a pretreatment step prior to undertaking further advanced treatment steps. Fine-solids removal is also often necessary as a "polishing" step after chemical or biological treatment processes. However, conventional physical treatment steps such as sedimentation or multimedia filtration, have inherent disadvantages: for example, the latter method often requires the addition of appropriate amounts of polyelectrolytes for coagulation, and skilled operators are usually required.

While conventional through-flow filters such as multimedia filters can be utilized for the removal of some suspended solids, they have the inherent disadvantage of requiring frequent backwashing, since the filtered particles continuously accumulate on, or actually enter, the filtration barrier. In other words, through-flow filtration is, by its very nature, a batch process, with the flux declining relatively rapidly when the driving pressure differential across the filtration barrier is held constant.

On the other hand, in cross-flow filtration the direction of the feed flow is parallel to the filter surface, so that accumulation of the filtered solids on the filter medium can be minimized by the shearing action of the flow. Thus, cross-flow filtration affords the possibility of a quasi-steady state operation with a nearly-constant flux when driving pressure differential is held constant. As discussed later, a flux decline over relatively long periods may still occur due to a variety of reasons.

The present paper describes a novel method of suspended solids removal utilizing cross-flow microfiltration with thick-walled, porous plastic tubes, called HYDROPERM™*. These tubes, whose wall thickness, inside diameter, porosity, and pore-size distribution can all be closely controlled, can be designed for any given effluent so as to obtain optimum performance. The filtration characteristics of the tubes combine both the "in-depth" filtration aspects of multi-media filters and the "surface" filtration aspects of membrane filters. For example, while the removal of micron-size particles and colloids is often impossible with conventional through-flow filters, the Hydroperm tubes are capable of removing such particles. However, in a manner similar to multi-media filters, the tubes may allow the smaller particles and colloids in the waste streams to actually penetrate into their wall matrix, without passing completely through.

Section II presents a brief discussion of the characteristic features of Hydroperm tubes and the basic aspects of cross-flow filter performance. This is followed in Sections III, IV and V by discussions of three applications of Hydroperm wastewater treatment for reuse.

The three applications discussed in the present paper are: (1) the treatment of wastewaters from the U. S. Army's "MUST" hospital complex; (2) the treatment of wastewaters from domestic and industrial laundries; and (3) the treatment of "pink water" wastes that are generated at Army Depots and at Army Ammunition Plants. The total Hydroperm "System" is discussed in Section VI.

SECTION II - CHARACTERISTIC FEATURES OF HYDROPERM™ TUBES

Hydroperm tubes, which can be made from a variety of extrudable thermoplastics, have several unique characteristics, including controlled microporosity and ruggedness.

A schematic view of cross-flow filtration through the tubes is shown in Fig. 1. The feed flow is through the inside of the tubes at relatively low pressures (5 to 10 psi) and the filtrate permeation occurs through the relatively thick (~ 1 mm) tube walls. Indeed, it is in the aforementioned two aspects that this type of microfiltration is significantly different from conventional membrane ultrafiltration, where higher filtration pressures (> 50 psi) are used and where permeation occurs through relatively thin membranes. Moreover, the pore structure of the Hydroperm tubes differs from those of membrane ultrafilters in that the pore sizes of the former are of the order of several microns with the "length" of the pores being many times their diameters.

---

* HYDROPERM™ is the proprietary name applied to microfiltration tubes developed by HYDRONAUTICS, Incorporated.

Pore size distributions of three typical tubes are shown in Fig. 2. Tube I has a rather "flat" distribution with the pores ranging in size from 2 to 10 microns. On the other hand, Tube III has a "peaked" distribution, with most of the pores being in the 2-micron range. Tube II has an intermediate distribution.

The tubes can be made from several thermoplastics, such as polyethylene and nylon. Tubes I and II in Fig. 2 are made from polyethylene, while Tube III is made from

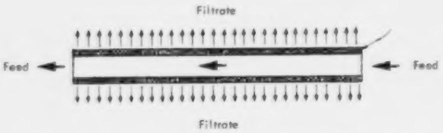


Figure 1 - Crossflow Filtration Schematic

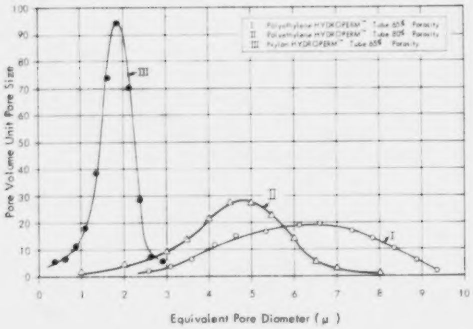


Figure 2 - Typical Pore Size Distribution of HYDROPERM™ Tubes

nylon. These features are of crucial importance in determining the performance of a given tube when it is used with a specific feed.

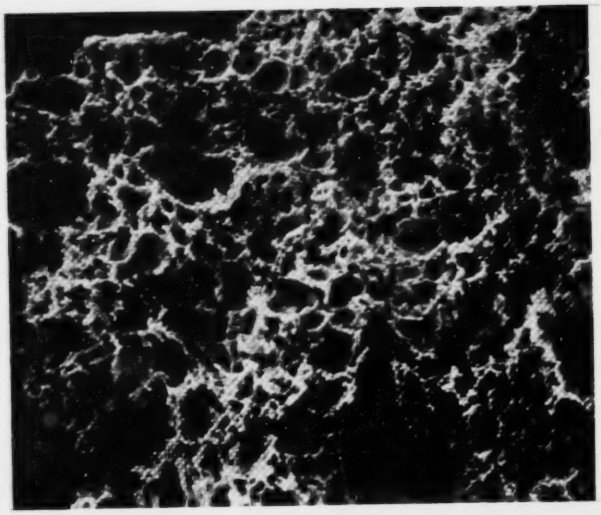
Two views of the pore structure of a typical tube are shown in Fig. 3. These photographs were taken with a Scanning Electron Microscope and are of a transverse section of the tubes; the view in (a) has a magnification factor of two hundred, while that in (b) has a magnification factor of one thousand. The open-cell, reticulated nature of the pore structure can be appreciated from these photographs.

BASIC ASPECTS OF CROSS-FLOW FILTER PERFORMANCE

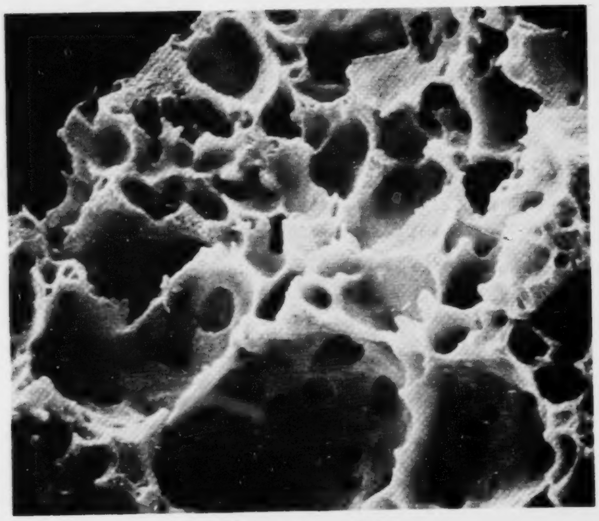
In general, filtration performance of a given tube with a specific waste effluent will depend both on the characteristics of the tube (such as porosity, pore structure, and pore-size distribution) and on the operating conditions (such as pressure, temperature, and cross-flow velocity). While the actual functional dependence of the filtration performance on the various parameters of relevance is quite complex, some general conclusions can, nevertheless, be drawn by considering a fairly simple model for the filtration process. In general, any wastewater from which suspended solids removal is desired will contain a wide range of particulates, ranging in size from several microns to colloidal dimensions. When such effluents are circulated through the inside of any tubular filter with porous walls, the solid particles will be slowly driven, with the permeating flow, toward the wall. Thus, the concentration of the particles in regions close to the wall will tend to steadily increase, this tendency being delimited only by the turbulent diffusion of the particles from regions of high concentration to those of lower concentration (i.e., away from the walls toward the center of the tube).

The turbulent diffusion (which tends to decrease the particle concentration near the wall) is dependent on the shear stress that is exerted on the walls by the cross-flow circulation and, hence, its velocity. On the other hand, the permeation rate (which tends to increase the particle concentration near the wall) depends on the pressure differential across the filter surface (Poiseuille's law) as well as the pore structure of the tubes (Darcy's law). A quasi-steady state profile of the concentration of the particles will be established near the wall when the two aforementioned opposing tendencies exactly balance each other. The resulting "particle polarization" in this case is entirely analogous to the "concentration polarization" of solutes that occurs close to walls of ultrafiltration and reverse-osmosis membranes.

Because of the in-depth filtration characteristics of the Hydroperm tubes, other factors also come into play. Specifically, particles which are smaller than the largest pore size of the tubes can actually enter the wall matrix, while particles which are larger than all of the pores in



a) S.E.M. 200X



b) S.E.M. 1000X

Figure 3 - Electron Microphotographs of HYDROPERM™
Tube Pore Structure - Transverse Section

the tubes will be retained at the walls. Thus, as filtration proceeds, the pore structure of the tube as well as its permeability will undergo a gradual change due to the penetration of some of the pores by the intruder particles. However, the tendency of new particles to enter the tube matrix will be decreased as a thin, dynamic filter cake forms on the walls due to the particle polarization described earlier. The flux decline due to these processes will be approximately exponential in character, with the characteristic time scale being a function of both the nature of the particles in the wastes being treated as well as the pore structure of the tubes.

Even from the relatively simple, qualitative discussion given in the foregoing, it is clear as to how the filtration performance is influenced by such factors as the filtration pressure, circulating flow velocity, temperature (which changes the fluid viscosity and, hence, by Darcy's law, the permeation rate), pore-size distribution, pore structure, and the particle-size distribution in the wastes. As mentioned earlier, the unique feature of the Hydroperm tubes is that their pore characteristics can be designed (i.e., controlled in a systematic manner) to suit the characteristics of a given category of waste effluent.

SECTION III - TREATMENT OF "MUST" HOSPITAL WASTEWATER EFFLUENTS

INTRODUCTION

The U. S. Army's mobile "MUST"[*] medical complex, which was recently under development, was designed for rapid establishment and disestablishment. An integral part of the "MUST" system was a Water and Wastewater Management Subsystem (WWMS), designed to treat all the wastewaters which are generated within the complex, with the treatment being of sufficient quality for some type of water reuse. Several unit treatment operations have already been studied extensively in terms of their ability, in various combinations, to treat wastewaters from operating rooms, X-ray, laboratories, laundries, showers, and kitchens. These treatment operations include microfiltration, ultrafiltration (UF) and hyperfiltration (reverse osmosis). Under the sponsorship of the U. S. Army Medical Bioengineering Research and Development Laboratory, HYDRONAUTICS, Incorporated recently investigated the feasibility of using Hydroperm microfiltration as a possible alternative to ultrafiltration. The specific objective of the study was not merely to test the "MUST" wastes with Hydroperm tubes, but to do so under rather extreme conditions of circulating feed pH and temperature. In the conduct of these investigations, a composite test fluid (Table 1) was chosen, based on the recommendation of the Bioengineering Laboratory. The resultant composite feed is not only a typical representative sample, but also gives emphasis to those constituents which might cause fouling of membrane filters.

---

[*] Medical Unit, Surgical Transportable

Table 1

"MUST" Hospital Composite Waste

	Constituent	Conc. (mg/ℓ unless otherwise stated)
1.	Detergent Type I (FSN 7930-634-3935)	221.0
2.	Sparklene (Fisher Chem.)	202.0
3.	Haema-Sol Detergent (nonsudsing)	197.0
4.	Hair	114.0
5.	Shower/Laboratory Cleaner Formula SBS-52	50.5
6.	Hand Soap (Lava)	34.8
7.	Scouring Powder	22.1
8.	Talc	10.1
9.	Soil (Kaolinite)	9.6
10.	Silver Chloride	7.2
11.	Hair Oil	75.8
12.	Hair Gel	18.7
13.	Vegetable Oil	17.6
14.	Grease (Lard)	11.7
15.	Toothpaste	18.7
16.	Hair Shampoo	2.5
17.	Suspended Solids (Dog Food)	140.0
18.	Blood (Animal)	183 µ 1/1
19.	1½% Agar	70 µ 1/1
20.	Betadine	189.0
21.	Wescodyne	35.6

The test program consisted of initial tests with 45 cm long single Hydroperm tubes at two (extreme) values of temperature and pH. Tubes of two different pore structures were used in the tests. Flux behavior and cleaning procedures for flux restoration were investigated during these tests, which were each approximately 50 hours in duration. After the initial tests, long-term tests were undertaken with a small module consisting of several Hydroperm tubes.

TUBE OPTIMIZATION TESTS

One of the principal objectives of this study was to optimize the pore structure and materials of the tubes for maximum performance. This optimization was carried out through laboratory tests on single tubes. A schematic view of a typical single-tube test loop is shown in Fig. 4. As indicated in the figure, the test loop contains a feed reservoir (~ 3 gal, 11.4 ℓ capacity), a circulating pump, a flow meter, pressure gauges to measure pressure drops over the length of the tube being tested, and appropriate valving. Provision also exists for controlling the circulating fluid temperature and velocity.

In the first test series, tubes of two different materials (nylon and polyethylene) and two different internal diameters (6 mm and 9 mm) were used. The test matrix was also composed of two values of pH (nominally 2 and 8) and two values of operating temperature (nominally 4°C and 50°C). In all of the tests the average filtration pressure was 5 psi and the circulating feed velocity was 7 ft/sec. The composition of the synthetic waste feed is shown in Table 1.

A typical test result from the tube optimization tests, including cleaning, is shown in Fig. 5 for a 6 mm ID nylon tube tested at 48.9°C and pH of 8.0. The initial flux of this tube was about 140 gal/ft^2-day (gfd), (5,700 liters/m^2-day). The flux declined to about 20 gfd (814 ℓmd) after 50 hours of continuous operation. The characteristic shape of the flux-time curve as shown in this figure indicates an initial relatively rapid decline followed by a nearly-constant plateau. After 50 hours of operation the tube was cleaned by circulating through it, for 10 minutes, a weak solution of phosphoric acid (available under the commercial name of Servac). It can be seen from the figure that this cleaning restored the flux to about 70 gfd. More importantly, after another 70 hours of operation, the flux had leveled off at about 45 gfd. This tube was operated for over 200 hours, with several cleanings, and with an average flux ~ 40 gfd. A 9 mm nylon tube tested at a temperature of 48.9°C and pH 7.7 had an initial flux of 58 gfd, plateauing at 10 gfd. The flux was restored to over 80 gfd and remained at > 30 gfd after cleaning with hypochlorite (Fig. 6). Tests at low temperature (4.4°C) had plateau fluxes in the range of ~ 10 gfd, as would be expected.

Based on all test results, the tube selected for use in a module for the "MUST" pilot plant was a 6 mm, nylon tube. The recommended operating conditions are 5 psi pressure and a circulating velocity of 7 ft/sec (or

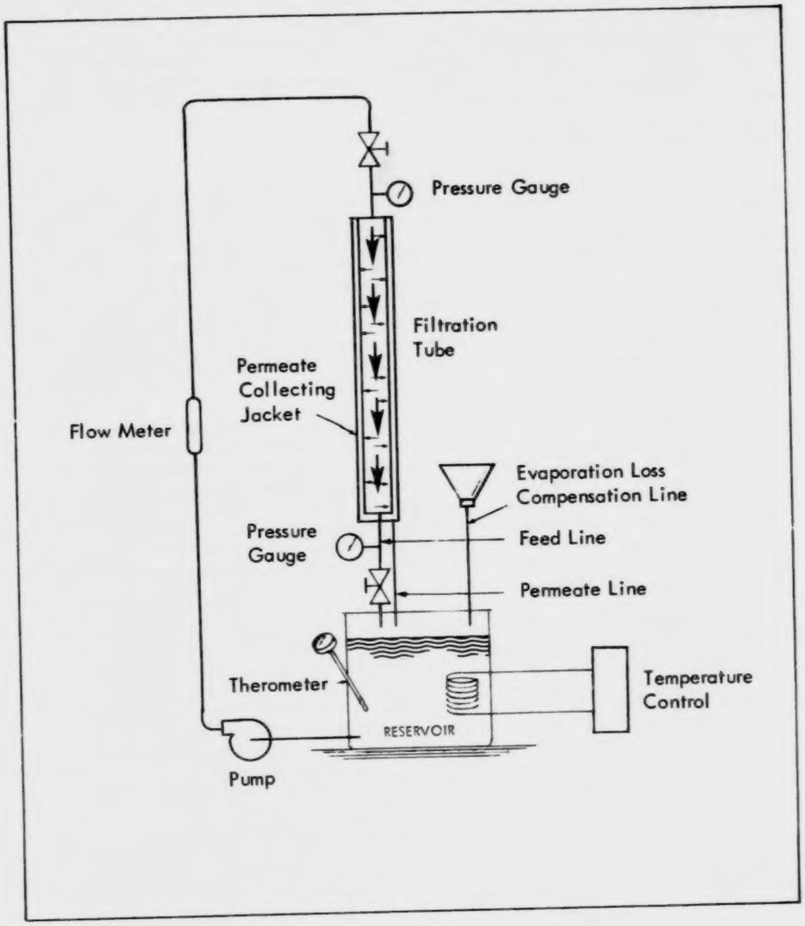


Figure 4 - Schematic of a Single Tube Test Loop

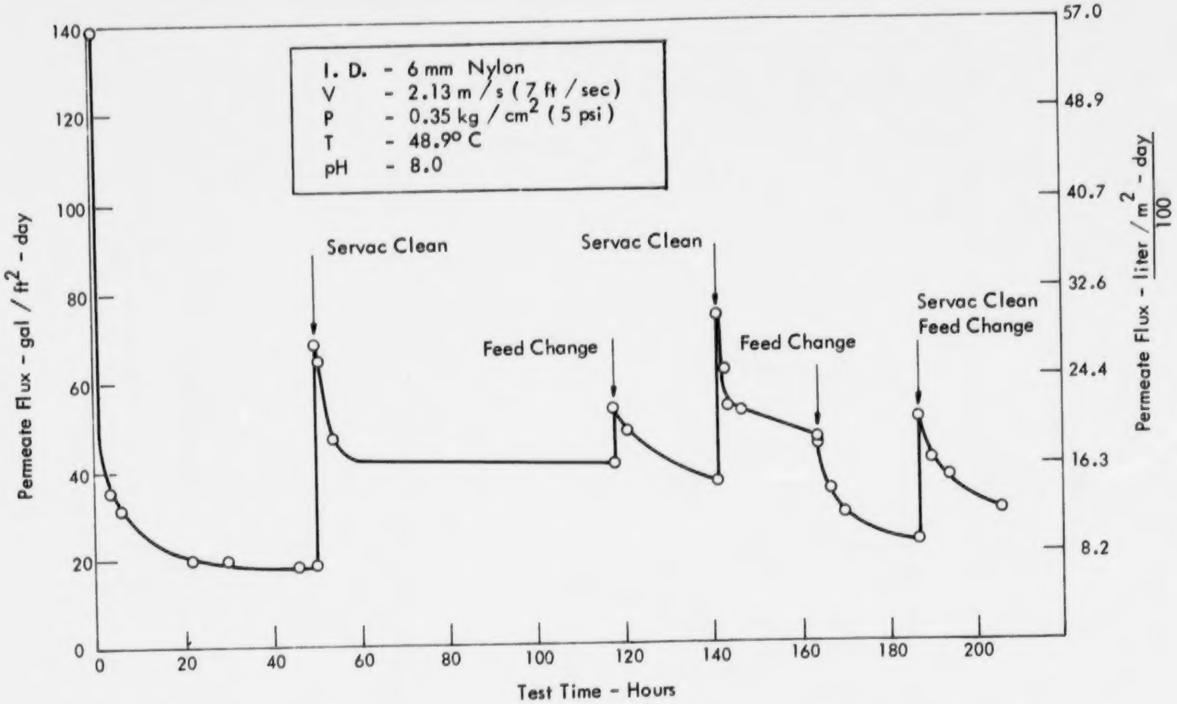


Figure 5 - Hydroperm Test on "MUST" Waste at High Temperature and Moderate pH

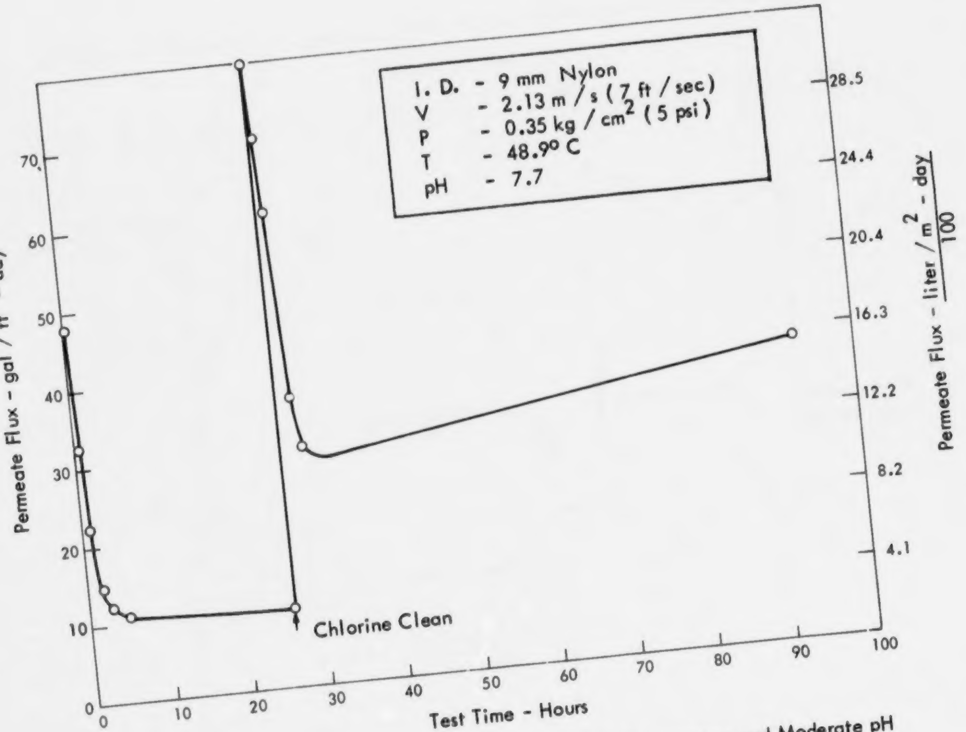


Figure 6 - HYDROPERM Test on "MUST" Waste at High Temperature and Moderate pH

approximately 1 gpm of circulating flow per tube).

TESTS ON LONG-TERM FLUX BEHAVIOR

After the initial tube-optimization tests and the selection of the appropriate tube for use with the MUST wastes, a test of several-hundred-hours duration was conducted to determine the long-term flux behavior of the selected tube. The results of the test are shown in Fig. 7. The test was conducted at room temperature, that is, at approximately 25°C; however, in Fig. 7 the flux data themselves as well as those corrected to a temperature of 48.9°C are shown, so as to afford direct comparison with the results shown in Fig. 6. It can be seen from the figure that after 328 hours of continuous operation, the nature of the flux behavior is essentially the same as that during the first few hours of operation.

Comparison of Figs. 6 and 7 show, as expected, considerable similarities in flux behavior. The comparison also reveals that the spuriously high value of the "plateau" flux noted after the first cleaning in Fig. 6 can be attributed, at least partially, to the change in feed characteristics due to repeated recirculation at a high temperature. The long-term tests also reveal that best results in flux restoration are achieved when the tubes are first cleaned for ten minutes with a hypochlorite solution, followed by cleaning for ten minutes with a Servac solution.

TESTS IN CONCENTRATION MODE

All of the tests described thus far were done in a constant-concentration recirculation mode; that is, the filtrate was continuously remixed into the feed holding tank, so as to keep the feed conditions as nearly constant as possible, throughout the tests for purposes of comparison. However, separate tests were also done on concentrating, to various degrees, a given initial volume of the waste; that is, in these tests the filtrate was removed to a separate reservoir and the solids concentration in the feed was allowed to increase continuously.

Results from the concentration mode test using a single tube of 6 mm ID and 38.5 cm length (filtration surface area = 72.6 cm^2) are shown in Fig. 8. These tests were conducted at a pH of about 8 and at a temperature of about 4.4°C. As in the constant-concentration mode of testing, the initial flux over the first few minutes of operation was high (about 60 gfd) (2400 ℓmd); and after four hours of uninterrupted operation, the flux had declined to about 23.5 gfd (950 ℓmd). During this time, about 0.56 gallons (2.12 liters) had been removed as a clear, colorless permeate from the initial 2 gallons (7.56 liters) of feed, for a volume reduction of 28%. The volume reductions are indicated in Fig. 8 by diamonds and the corresponding values are shown in the right-hand scale.

After four hours of operation, the test was stopped

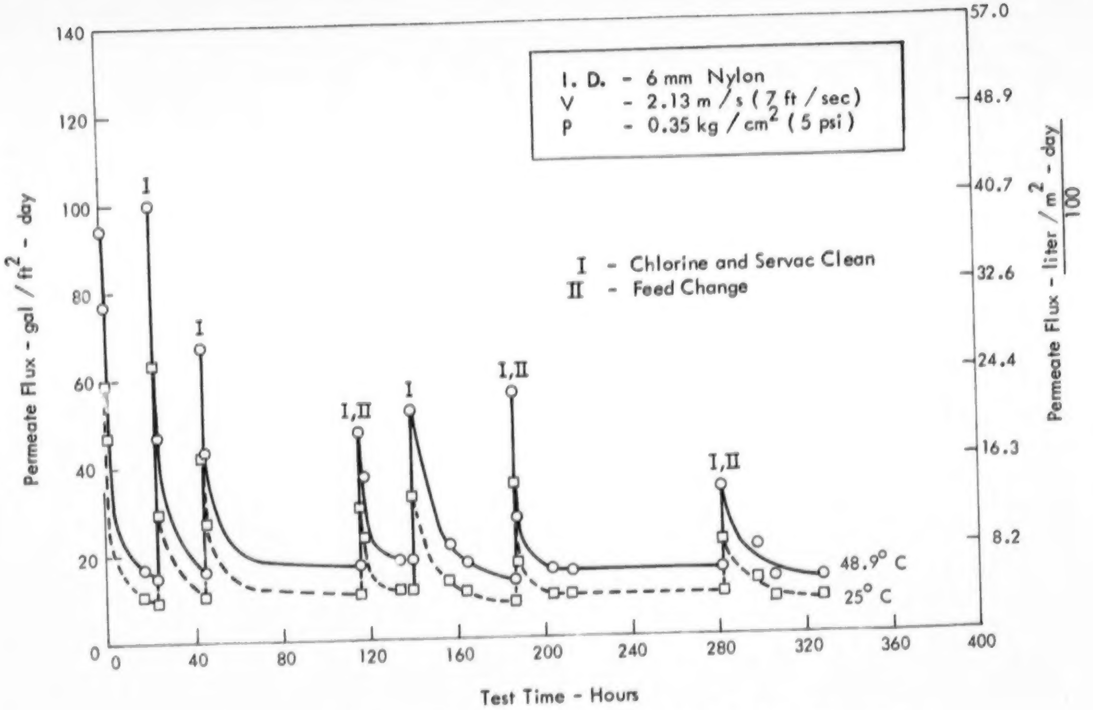


Figure 7 - HYDROPERM Tests on "MUST" Waste Long-Term Flux Behavior

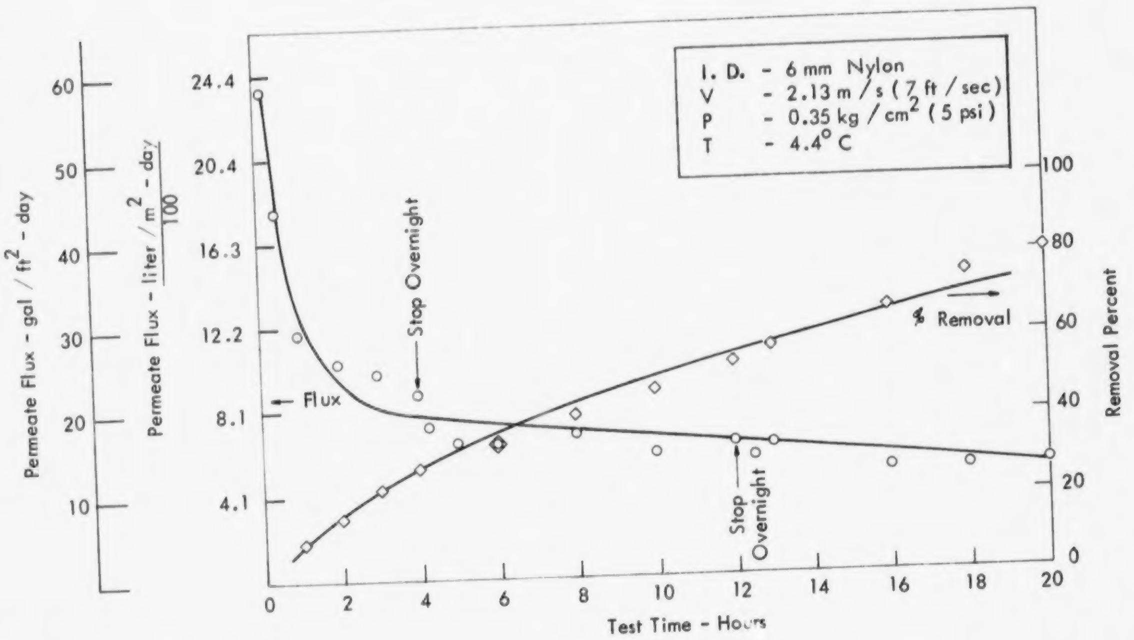


Figure 8 - HYDROPERM Concentration Test on "MUST" Waste at Low Temperature

overnight. However, at the resumption of testing the following morning, the fluxes were essentially the same, indicating that the stagnant conditions did not adversely affect filtration performance. After twenty hours of operation, eighty percent of the initial volume of the feed had been removed as permeate. It can be seen from the figure that the flux after twenty hours is only slightly lower than at four hours in spite of the five-fold increase in concentration. Since 1.61 gallons (6.09 liters) were removed in 20 hours, and since the filtration area of the tube was 0.078 ft^2 (72.6 cm^2), the average flux during this period was 24.8 gfd (1,000 ℓmd).

A second concentration-mode test was conducted at room temperature (~ 25°C) using a small module which had a total of 2.31 ft^2 (2143 cm^2) of filtration area. The results of this test are shown in Fig. 9, both in terms of the flux and the degree of volume reduction. The values of the flux adjusted to 48.9°C are also shown in the figure. The initial volume of feed in the reservoir was 10 gallons (37.85 liters) and, as the concentration proceeded, fresh batches of feed were added to the reservoir, so that at the end of 3½ hours of operation the total volume of feed processed was 25 gallons (94.64 liters). During this period 24 gallons (90.85 liters) of filtrate were collected as a clear, colorless fluid, so that the volume reduction achieved was 96%. Since 24 gallons (90.85 liters) of permeate were collected during 3.5 hours, using a filter area of 2.31 ft^2 (2143 cm^2), the average flux during the concentration process was 71.24 gfd (2900 ℓmd).

The volume of feed remaining after 3½ hours of operation was not sufficient to enable maintenance of pump suction; hence, the test was stopped. However, the one gallon (3.79 liters) of concentrate remaining was further processed in a smaller, single-tube test loop, and the results are shown in Fig. 10. After seven hours of operation, a further 0.66 gallons (2.5 liters) of filtrate were collected using a tube having a surface area of 0.078 ft^2 (72.6 cm^2). Thus the average flux during this phase of the test was 29 gfd (1179 ℓmd).

If the filtration area, time and average flux during the first and second phases are, respectively, A_1, t_1, q_1 and A_2, t_2, q_2, then the average flux, $\overline{q}$, is equal to $(A_1t_1q_1 + A_2t_2q_2)/(A_1t_1 + A_2t_2)$ = Total Volume of Filtrate$/(A_1t_1 + A_2t_2)$.

The average flux over the entire concentration period can be computed to be 68.52 gfd (2790 ℓmd).

The tests described above demonstrate that Hydroperm filtration can achieve large volume reductions of hospital wastewater (nearly seventy-fivefold concentration) at significantly high flux levels.

PERMEATE QUALITY ANALYSIS

During the constant-concentration tests, permeate and feed samples were collected and analyzed for total solids, suspended solids, BOD and turbidity. The results are

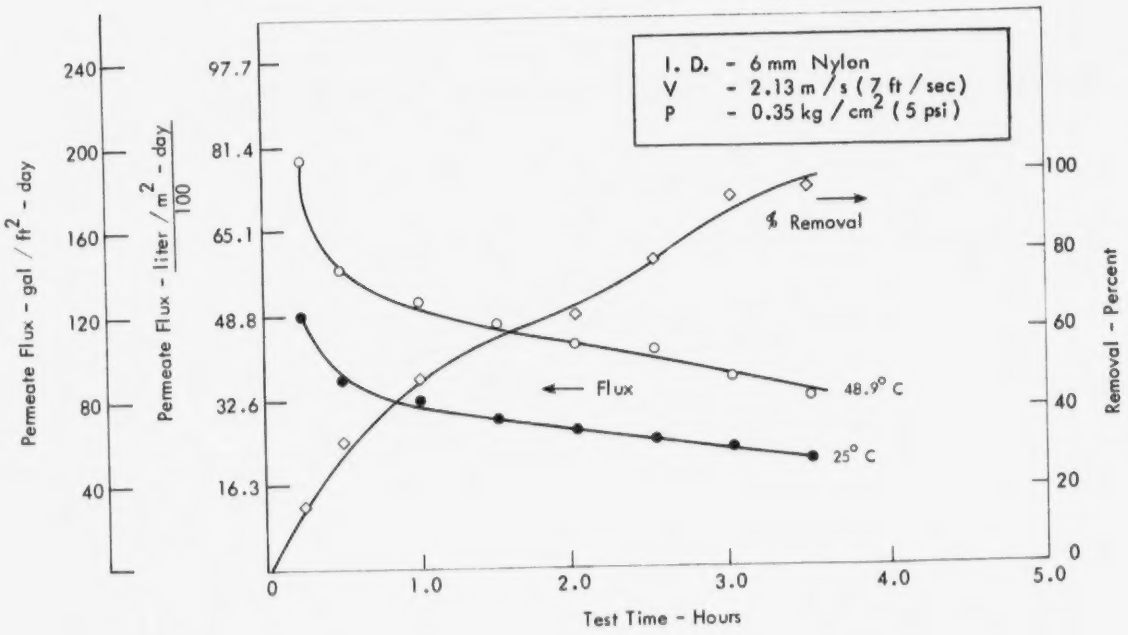


Figure 9 - HYDROPERM™ Concentration Test on "MUST" Waste at Room Temperature

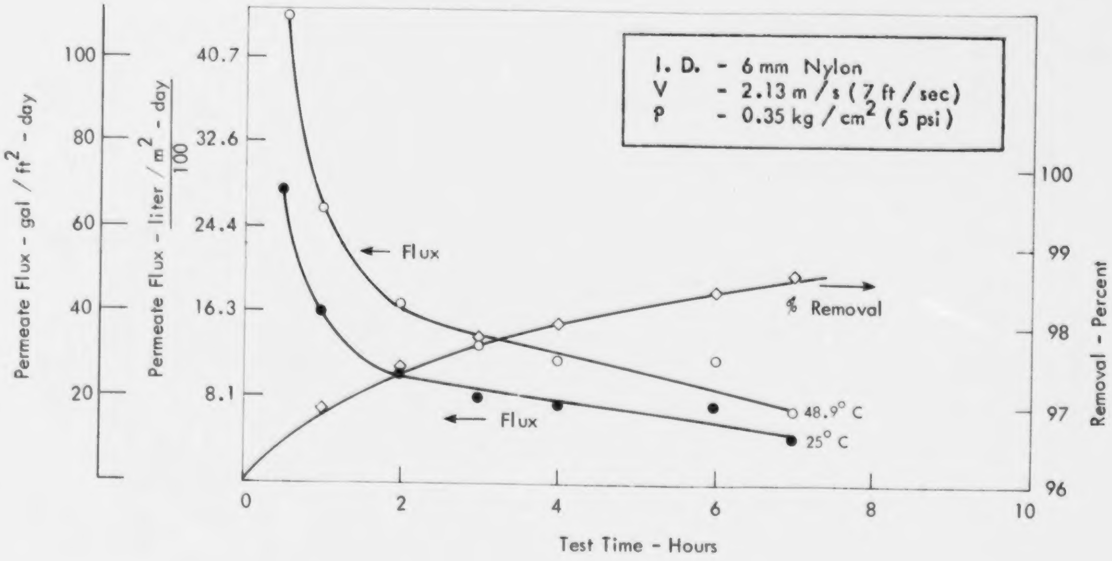


Figure 10 - HYDROPERM Concentration Test on "MUST" Waste at Room Temperature - Single Tube

listed in Table 2 below.

Table 2

Analysis of "MUST" Waste

	Feed	Permeate	% Removal
Total Solids (mg/ℓ)	1,200	490	59.2
Suspended Solids (mg/ℓ)	190	0	100.0
Dissolved Solids (mg/ℓ)	1,010	490	51.2
BOD (mg/ℓ)	135	20	85.2
Turbidity, JTU	98	0.24	—

On-site tests with shower water conducted in 1978 by the U. S. Army Medical Bioengineering Research and Development Laboratory at Fort Detrick showed that Hydroperm microfiltration achieved solids rejection comparable to ultrafiltration membranes, as shown in Table 3. Laboratory as well as ocular and dermal analyses showed that the permeate was of good enough quality for reuse as shower water after chlorination.

Table 3

Shower Wastewater Permeate Quality

	TOC	COD	TS	Turbidity
Feed	142	523	883	250
Permeate	29	76	475	0.5

As in tests with other waste effluents, Hydroperm filtration achieved total removal of suspended solids from the "MUST" waste. The high dissolved solids rejection is also typical of Hydroperm filtration, and has to be attributed to the in-depth nature of the filtration process as well as to the formation of a "dynamic membrane" on the inside surface of the tube.

It can also be seen from Tables 2 and 3 that there is significant reduction in BOD, TOC and COD and an almost total removal of turbidity. Indeed, the turbidity of the permeate was often indistinguishable from that of the control sample of distilled water. It is relevant to note that the near-total removal of turbidity and suspended solids was maintained even when concentrated feed wastes were filtered (during the concentration mode of testing).

SUMMARY OF THE "MUST" RESULTS

The laboratory test results described above demonstrate that Hydroperm tubes are capable of effectively filtering "MUST" hospital wastewaters even under extreme conditions of pH and temperature without irreversible fouling or clogging. The flux-time curves for all of the test conditions exhibit a characteristic behavior in which the initial flux declines within the first few hours of operation to a nearly-constant, "plateau" value. The flux is restored to the initial value when a fairly simple cleaning procedure is used. Under this procedure a weak hypochlorite solution is first circulated through the tubes (at the same conditions of pressure, temperature and flow velocity as in the filtration operation) for ten minutes, followed by a similar treatment with a Servac solution.

From the continuous-mode (constant-concentration mode) test results, it can be seen that even under rather extreme conditions of pH and (low) temperature, Hydroperm yields time-averaged fluxes over twenty-four hours of operation of at least 22.5 gfd (900 ℓmd). In other words, this level of flux can be maintained by cleaning the tubes once every twenty-four hours or so. The tests in a concentration mode yield average values of the fluxes which are even higher. Specifically, the time-averaged flux at 4.4°C is 24.8 gfd (1,000 ℓmd), while at 48.9°C it is 68.5 gfd (2790 ℓmd).

SECTION IV - TREATMENT OF LAUNDRY WASTEWATER EFFLUENTS

INTRODUCTION

There has been a significant amount of effort directed by the U. S. Environmental Protection Agency (Table 4) toward characterization of the major constituents of wastewaters of commercial and industrial laundries, and toward development and evaluation of treatment technologies to remove objectionable constituents. Two major potential objectives of the treatment of laundry wastewaters are savings from water reuse and energy savings from reuse of treated, hot wash and rinse waters. In addition, there would be significant environmental advantages. These laundries receive a wide variety of textiles soiled from both domestic and industrial activities. Wastewater parameters such as detergents, phosphates and other detergent builders, oil and grease, suspended solids and toxic heavy metals have received world attention. In view of the great variety of laundry wastewaters and the ability of the Hydroperm tubes to be "tailored" to different wastewaters, considerable testing has been carried out by HYDRONAUTICS in the laboratory to determine long-term flux behavior, as well as the effect of concentration on permeate quality.

Two studies are discussed in this section: domestic and industrial laundry wastes. The Hydroperm tube selection process for these particular wastewaters is similar to that for the "MUST" wastes discussed in the previous Section. Only the discussion of the test results are presented below.

DOMESTIC LAUNDRY WASTEWATER TREATMENT

The results of a test with domestic laundry wastewater at 35°C and for a duration of 500 hr are shown in Fig. 11. These laundry wastes were obtained from the U. S. Naval Academy Laundry, Annapolis, Maryland. The initial flux at the start of the tests was 100 gfd (4070 ℓmd), and after 92 hours of continuous operation, it fell to 51 gfd (2096 ℓmd).

Table 4

Characteristics of a
Cincinnati Industrial Laundry Wastewater

Parameter	Range	Average
pH	9.2-10.2	—
BOD, mg/ℓ	1450-8380	4930
COD, mg/ℓ	6590-33,900	20,200
TOC, mg/ℓ	855-4290	3000
Total Solids, mg/ℓ	2540-15,100	8013
Suspended Solids, mg/ℓ	905-7870	4020
Grease, mg/ℓ	1800-6000	3900
Hg/μg/ℓ	not analyzed	—
Cr, mg/ℓ	0.4-5.0	2.0
Cu, mg/ℓ	0.9-5.0	3.0
Pb, mg/ℓ	6.0-30.0	20.0
Zn, mg/ℓ	3.0-20.0	10.0
Cd, mg/ℓ	0.1-0.8	0.4
Ni, mg/ℓ	<0.7	—

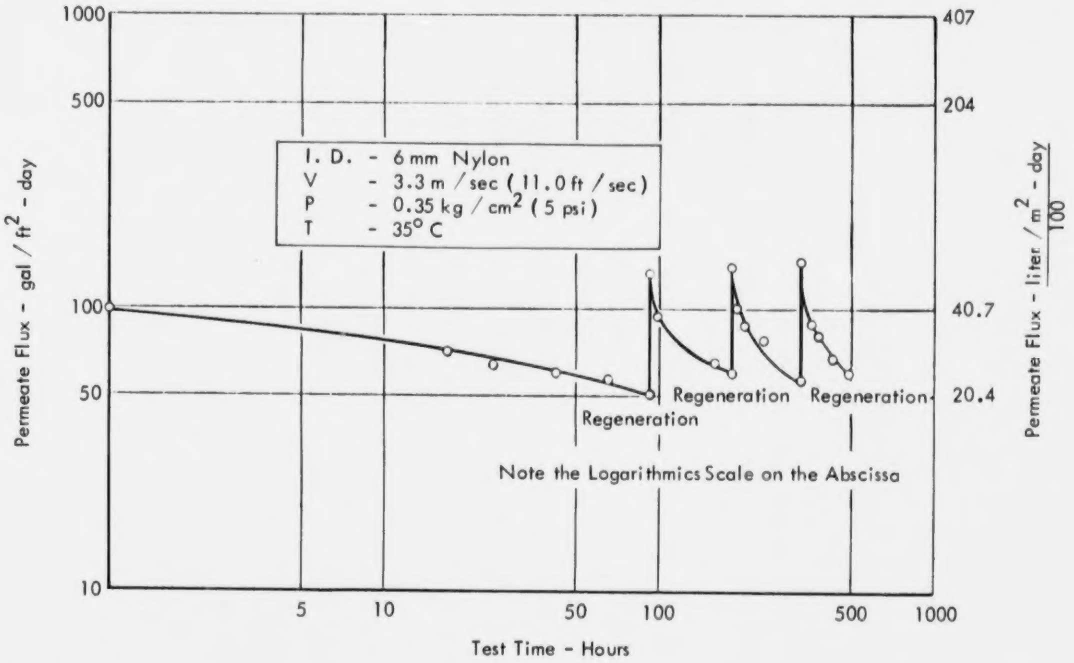


Figure 11 - Treatment of Domestic Laundry Wastes -- Continuous Mode Operation

At this point, circulation of the laundry waste was stopped and the tube was "cleaned" for 10 min. by circulating through it, at the same operating conditions as in the test, a weak solution of Servac (a mild phosphoric acid solution). When the test was resumed with the laundry waste, the initial flux was 155 gfd, (6320 ℓmd). The flux then gradually decreased as shown in Fig. 11 and, after 92 hr of further continuous operation (i.e., after a total of 184 hours after the start), fell to 60 gfd (2442 ℓmd).

As shown in the figure, the test was continued over a period of 500 hours with periodic cleaning using Servac. It can be seen that each cleaning essentially restored the flux to about 155 gfd (6310 ℓmd) and that after about 100 hours of continuous operation, the flux again decreased to about 60 gfd (2442 ℓmd). Thus, the test demonstrated that with domestic laundry wastewater average flux levels of over 80 gfd (3256 ℓmd) can be maintained virtually indefinitely by using the simple cleaning procedure for 10 min. every 100 hours. The test described in the foregoing was performed in the "constant-concentration" mode; i.e., the feed concentration of the suspended solids was maintained constant throughout the tests.

Another test was performed to study the effect of operating in a "concentration mode" in which the filtrate is removed to a separate holding tank; in this case, the suspended-solids concentration in the feed increases steadily and the total volume of the circulating fluid decreases steadily. The results of this test are shown in Fig. 12. The test was begun with 4.85 gal (18.3 ℓ) of the feed in the holding tank. After nearly 13 hours of continuous operation, only 0.16 gal (0.6 ℓ) of feed remained in the reservoir. The reduction in volume of the feed during the test is shown in Fig. 12, with the right-hand scale representing percentage reduction in volume. The permeate flux at the start of the test was 144.5 gfd (5880 ℓmd), while at the end of the test (12 hours) it was 89.5 gfd (3642 ℓmd).

Feed and permeate samples were collected at several points in the tests, and the results of the analysis are given in Table 5, with the numbers being keyed to the ones indicated in Fig. 12 for the time at which the samples were withdrawn. Both measured and calculated values are shown in Table 5. Several interesting aspects of the filtration are evident from examining the table. First, the suspended solids concentration in the permeate is zero regardless of the concentration in the feed! Second, the tubes display a finite rejection for dissolved solids! Note that the amount of dissolved solids in both the feed and permeate can be calculated simply by taking the difference between the total and suspended solids. Such finite rejection for dissolved solids, while surprising for a filter with pore sizes in the micron range, is nevertheless typical in Hydroperm filtration, as demonstrated for the "MUST" and Pink Water wastes. In the present case, the dissolved-solids rejection is about 35 percent.

As already pointed out, the finite dissolved-solids rejection has to be attributed to the thin filter cake

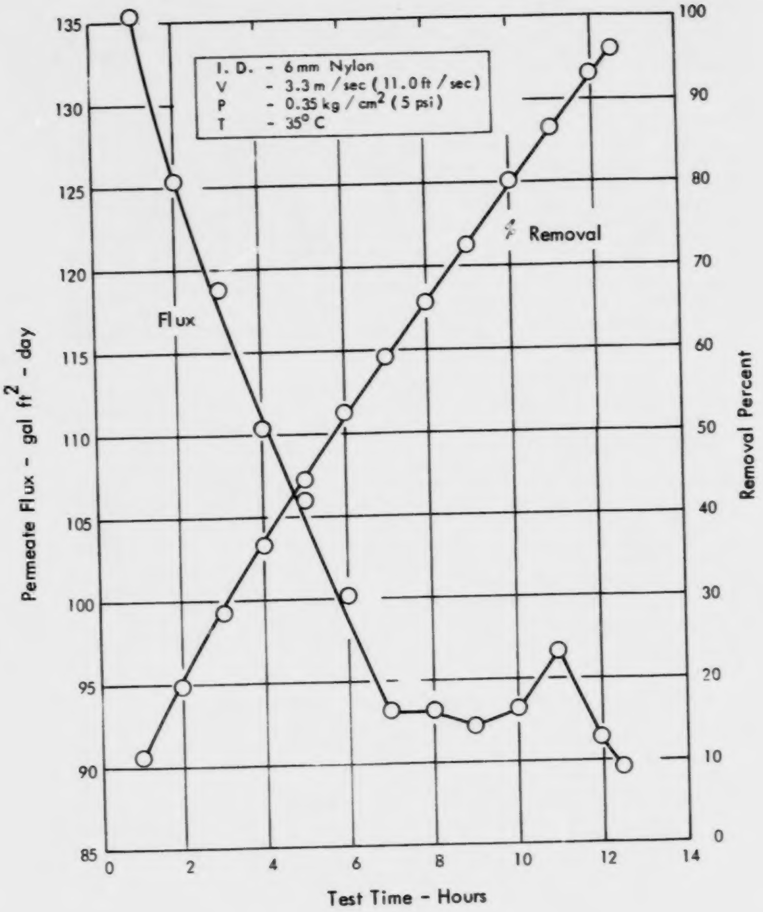


Figure 12 - Treatment of Domestic Laundry Wastes - Concentration Mode of Operation

(which acts as a "dynamic membrane") which develops on the inside surface of the tube walls, and to the changes in the pore structure of the tube due to the partial intrusion into it of some of the fine solids present in the feed.

The initial suspended-solids concentration in the waste was about 160 mg/ℓ. It can be seen from Table 5 that the suspended solids content in the final feed (about a 25-fold increase over the initial value) is consistent with the 96 percent volume reduction achieved. The fact that there is an increase in concentration of the dissolved solids in the feed is a direct consequence of the finite rejection displayed by the tubes. Also, the near-constancy of the COD levels in the permeate seems to indicate that the COD which gets through the tubes is that part associated with dissolved solids alone.

INDUSTRIAL LAUNDRY WASTEWATER TREATMENT

Wastewaters from two different industrial laundries were also tested. One of the laundries is in North Carolina, while the other is in the Chicago area. Both laundries process work uniforms and various types of industrial wipes, towels, linens, rags, mops, carpets, etc., so that, when compared to the Naval Academy laundry wastes tested earlier, these effluents had considerably higher waste loadings. For example, the waste from the North Carolina laundry had significantly more suspended solids (about 400 mg/ℓ compared to less than 100 mg/ℓ for the Naval Academy wastes), more total solids (over 3000 mg/ℓ compared to less than 1500 mg/ℓ) and more COD (nearly 4000 mg/ℓ compared to about 400 mg/ℓ). The wastewater from the Chicago laundry had even larger waste loadings, with suspended solids about 5600 mg/ℓ and total solids about 41,000 mg/ℓ! Both wastewaters also contained appreciable amounts of oil with the wastewater from the North Carolina laundry containing about 350 mg/ℓ and that from the Chicago area containing nearly 15,000 mg/ℓ.

Both laundries discharge their effluents into municipal sewers. However, because of the increasingly stringent restrictions which are being placed on wastes which are discharged into the sewers, and because of the penalties that may be imposed when the TS, the BOD, and heavy metals of the effluents exceed certain limits, there is considerable interest in the "pretreatment" of the effluents to a sufficient degree as to permit water reuse or to afford their discharge into the sewers. It is primarily for this reuse application that microfiltration was examined, and the quality of the permeate produced is sufficiently good so that it can either be discharged into the environment or be reused with little or no further treatment.

The filtration tests on the wastewaters from the North Carolina laundry were conducted with the same type of 6 mm ID, nylon tube that was used in the tests with the wastes from the Naval Academy laundry. The first test was done in a constant-concentration mode; that is the permeate was continuously remixed into the feed reservoir, so that the feed conditions were kept as nearly constant as possible

TABLE 5 - RESULTS FROM CONCENTRATION TESTS ON LAUNDRY WASTES

Sample Number	MEASURED VALUES						CALCULATED VALUES					
	Total Solids*		Suspended Solids*		COD*		Dissolved Solids*		Rejection Percent			
	Feed	Permeate	Feed	Permeate	Feed	Permeate	Feed	Permeate	Suspended Solids	Dissolved Solids	Total Solids	COD
1	1550	860	180	0	389	164	1370	860	100	37.2	44.5	57.8
2	2020	1010	370	0	619	146	1650	1010	100	38.8	50.0	76.4
3	2420	1270	460	0	947	155	1960	1270	100	35.2	47.5	83.6
4	2480	1280	490	0	1088	155	1990	1280	100	35.7	48.4	85.8
5	4580	1380	1980	0	3451	195	2600	1380	100	46.9	69.9	94.5
6	7010	-	3810	-	5972	-	3200					

* All values are in mg/ℓ

over the duration of the test. The single tube that was used during the tests was about 46 cm long and had an active filtration area of 11.25 in.^{2} (72.6 cm^2). The filtration pressure was 5 psi and the circulation velocity through the tube was 11 ft/sec.

The test results are shown in Fig. 13. The initial flux was about 62 gfd (2523 ℓmd) and after 164 hours of continuous operation, the flux had declined to 18 gfd (732 ℓmd). The average rate of flux decline during the first three hours is 7.63 gfd/hr (310 ℓmd/hr), while the average flux decline for the next twenty hours is only about 1.6 gfd/hr (65.1 ℓmd/hr). The average flux decline between eighty and one hundred and sixty hours of operation is only 0.08 gfd/hr (3.3 ℓmd/hr)! This behavior of an initially rapid flux decline and an eventual attainment of a "plateau" is a characteristic feature of cross-flow microfiltration.

The usual cleaning procedure was used in the present case after 164 hours of continuous operation. It can be seen from Fig. 13 that the cleaning increased the flux only to 23.9 gfd (972 ℓmd), a value which is well below the initial flux of 62.5 gfd (2543 ℓmd). Next, cleaning was attempted by circulating through the tubes, for ten minutes, a detergent solution. The flux now increased to 30 gfd (1221 ℓmd). The limited success achieved with the above two cleaning procedures is attributed to the presence of significant amounts of oil in the wastes and to the inability of the cleaning solutions to handle this oil.

Based on the consideration stated above, an enzyme detergent was next circulated through the tubes for ten minutes. As expected, there was a significant increase in flux to 37.4 gfd (1522 ℓmd) and, though the flux declined to about 26 gfd (1058 ℓmd) after three hours of operation, it remained essentially constant at this value for the next sixty-four hours of continuous operation! At this point the tube was once again cleaned with the enzyme detergent. The initial flux after cleaning was 47.2 gfd (1921 ℓmd) and after an additional twenty-two hours of operation, the flux was 34.5 gfd (1400 ℓmd).

The reason for successively higher "plateau" values of the flux after the first and second enzyme cleaning (when compared to the plateau flux before cleaning) is believed to be the degradation of the feed wastes, both due to natural aging and to the constant recirculation through the test loop. Such degradation is, of course, unavoidable when conducting laboratory tests with limited amounts of biologically and chemically active wastes. Thus the test was discontinued after a total of 276 hours of operation.

As mentioned earlier, the test just described was conducted at a constant feed solids concentration. To examine the effects of increased solids concentration in the feed, another test was conducted. The used feed from the first experiment was remixed with the remaining unused feed that was available (for a total volume of about 2 gallons), and the mixture was concentrated tenfold. The resulting suspended solids concentration in the feed was nearly 3,000

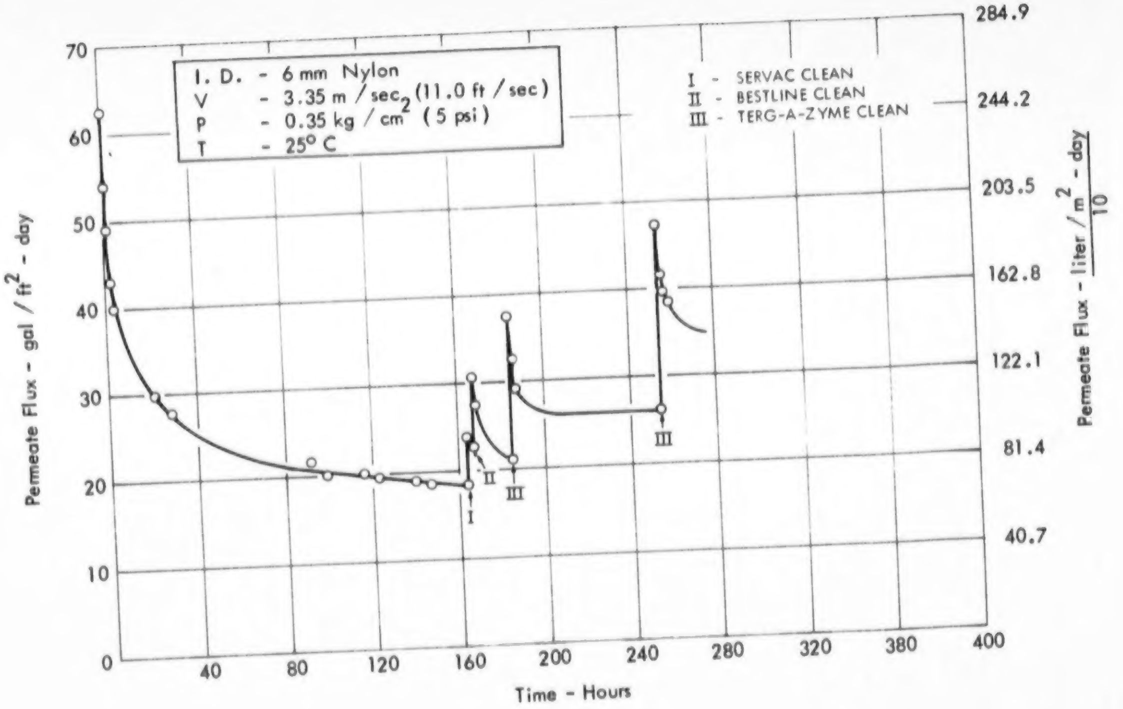


Figure 13 - Tests on Industrial Laundry Wastes (N.C. Laundry)

mg/ℓ. The concentrated feed was then tested with the same tube that was used in the first experiment; the results are shown in Fig. 14. The filtration pressure was again 5 psi, but the filtration velocity was reduced to 7 ft/sec. Since the power consumed in any given application is proportional to the cube of the velocity, it is advantageous to operate at as low a value as is compatible with good performance. It can be seen from the figure that in spite of the increased suspended solids concentration and the reduced circulating velocity, the filtrate flux quickly approaches a value of about 40 gfd (1628 ℓmd) which is essentially the same as that at the end of the first test.

Based on the tests described above, it can be concluded that Hydroperm cross-flow microfiltration can be quite effective for treatment of industrial laundry wastes. Adequate levels of flux can be maintained by occasional cleaning (for ten minutes once per day) with an enzyme detergent.

To further investigate the potential of treatment of laundry wastewaters for reuse, brief tests were also conducted, both in the constant-concentration and increasing concentration modes with the waste from the Chicago laundry. The flux results are shown in Fig. 15. The tests were conducted with a new section of the same type of tube that was used in the earlier tests. It can be seen from the figure that in spite of the very large amount of suspended solids (> 5500 mg/ℓ) and oil (15000 mg/ℓ) in the wastes, the average flux over forty-eight hours of uninterrupted operation was well in excess of 20 gfd (814 ℓmd). During a weekend of unattended operation, one of the valves in the test loop had clogged, causing a drastic reduction in the filtration pressure and hence the flux. However, cleaning of the tube, using the usual procedure with the enzyme detergent, substantially restored the flux. Beyond this point the filtration was continued in a concentration mode and flux levels of over 20 gfd (814 ℓmd) were maintained while removing 82% of tne circulating fluid as clear permeate. This test had to be discontinued only because of the limited amount of waste available, and the difficulty in maintaining pump suction with small amounts of the concentrate.

INDUSTRIAL LAUNDRY WASTE PERMEATE QUALITY

The analyses of the feed and the permeate from the tests discussed from the North Carolina laundry are shown in Table 6. It can be seen that the suspended solids are removed completely, while the (Freon extractable) oils as well as the BOD are removed almost entirely. There are also significant reductions in COD (84%) and dissolved solids (54%).

Analytical results of the feed and the permeate for the Chicago laundry waste tests are given in Table 7. It can be seen that in addition to the nearly total removal of suspended solids and (Freon extractable) oils, the dissolved solids are reduced by nearly 65%!

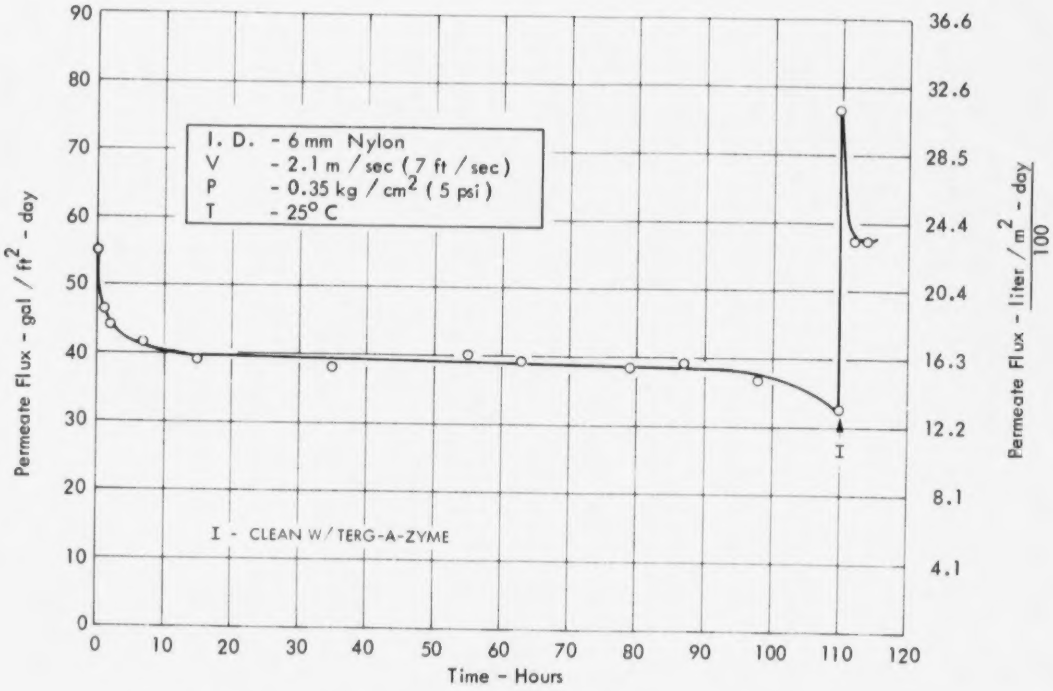


Figure 14 - Tests on Concentrated Industrial Laundry Waste (90% Concentration)

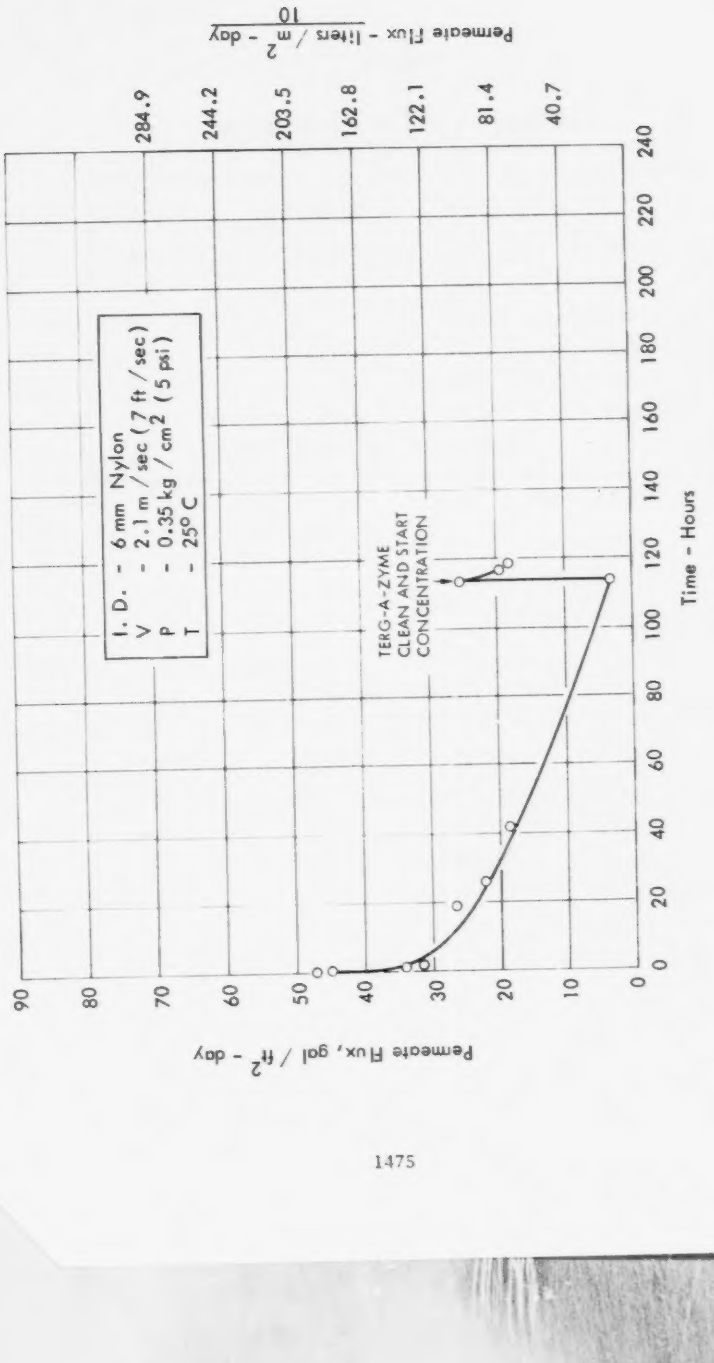


Figure 15 - Tests on Industrial Laundry Wastes (Chicago Laundry)

Table 6

Analysis of the Waste from the North Carolina Laundry

		Feed	Permeate	% Removal
TS	mg/ℓ	3268	1411	57
SS	mg/ℓ	290	0	100
Oil	mg/ℓ	341	3	99
DS*	mg/ℓ	3078	1411	54
BOD_5	mg/ℓ	250	10	96
COD	mg/ℓ	3820	607	84
pH		11.1	9.6	—

* Calculated

Table 7

Analysis of the Waste from the Chicago Laundry

		Feed	Permeate	% Removal
TS	mg/ℓ	40,857	7,535	81.5
SS	mg/ℓ	5,569	28	99+
Oil	mg/ℓ	14,246	83	99+
DS	mg/ℓ	21,042	7,424	64.8
pH		11.6	10.3	—

CONTINUING INVESTIGATIONS

In addition to the aforementioned tests, on-going investigations are being pursued to evaluate the feasibility of reuse of hot wash and rinse water in laundry operations. Tests will be conducted over a period of approximately six months. Initial results show virtually complete removal of suspended solids, oil and grease, as well as substantial reduction in dissolved solids. Thus far, it appears that it will probably be practicable to reuse laundry wastewaters treated only by Hydroperm to wash various types of loads in laundry operations.

SECTION V - TREATMENT OF MILITARY EXPLOSIVES WASTEWATERS

INTRODUCTION

The United States Army for a number of years has been implementing a program for the control of pollutants generated at military manufacturing facilities, such as Army Ammunition Plants and at Army Depots. The Army also has been assessing various technologies available for the abatement of pollution in these plants. In these assessments, it is recognized that not only must wastewater effluents be properly treated to produce an effluent acceptable to the environment, but that there is also a need for water management in a variety of scenarios, especially at manufacturing facilities and depots where water recycle and reuse is important.

A specific problem is the "pink water" which is generated in LAP (Load, Assemble and Pack) operations at Army Ammunition Plants, as well as in washout demilitarization operations at Army Depots. The pink water contains not only suspended and dissolved explosives, including TNT, RDX and others, but also contains complex nitrobodies which are the result of photolysis reactions upon exposure of TNT to sunlight. The pink water constitutes a high TOC and toxicity problem which currently requires expensive treatment with multi-media filters and with activated carbon. In addition, the pink water contents produce suspended solids upon cooling, with the resultant clogging of nozzles used in water wall spraying for dust removal. This inhibits the recycle of the pink water with conventional filtration. Moreover, the fine suspended solids can also lead to clogging of the carbon columns, thus necessitating more frequent and expensive recharging.

It is clear that an ideal treatment system would be one which maximizes the recycle and reuse of pink water and minimizes any carbon treatment requirements. This Section describes the results from the first phase of a multi-phase study whose objective is the development and incorporation into the Army wastewater treatment systems of an advanced filtration-separation subsystem, employing filtration by the Hydroperm microporous tubes.

The scope of the first phase of the pink water treatment program involved the conduct of laboratory filtration tests with typical pink water samples utilizing various types of Hydroperm tubes. As pointed out in Section II, one of the unique features of Hydroperm tubes is that their pore structure can be "matched", through systematic testing, to the characteristics of a given effluent. Thus, in the first task of this program, the optimum tube characteristics were identified for treating pink water effluents, by performing tests on several different types of tubes. The various tube characteristics included: tube internal diameter (4 mm, 6 mm and 9 mm); tube material (nylon and polyethylene); porosity (50 and 65 percent void volume); and pore sizes (median pore sizes ranged from about 10 μ to 1.5 μ).

PRELIMINARY HYDROPERM™ TEST RESULTS ON PINK WATER

The flux results from a typical test on a 9 mm nylon tube are shown in Fig. 16. It can be seen that the initial filtrate flux of this tube was in excess of 250 gfd (10175 ℓmd), although within about twenty hours of continuous operation the flux had attained a nearly level "plateau" value of about 70 gfd (2849 ℓmd); beyond this point the flux changed only gradually and after 110 hours of continuous operation, the flux was still over 60 gfd (2442 ℓmd). As has been pointed out previously, this two-stage flux behavior, consisting of a relatively-short duration initial period of rapid flux decline followed by a second stage in which the flux is nearly level, is typical of most Hydroperm applications examined to date. These flux levels can be maintained with only occasional (once a day) simple cleaning procedures.

A number of cleaning tests were carried out in this pink water study. The results from one such test are indicated in Fig. 17. In this test, after seventy-one hours of continuous operation, the flux had declined from an initial value of nearly 300 gfd (12210 ℓmd) to about 120 gfd (4884 ℓmd). At this point, the filtration operation was stopped and the cleaning operation was begun. First, a solution containing 10 mℓ/liter of hypochlorite was circulated through the tubes for ten minutes. This was then followed by a circulation for ten minutes of a solution containing 10 mℓ/liter of Servac.

When the filtration tests were resumed (with the same pink water wastes that were used before cleaning), the permeate flux had been increased to over 300 gfd (12210 ℓmd). More importantly, it can be seen from Fig. 17 that the flux-decline behavior following cleaning is nearly the same as that with the fresh tube. The filtration was continued uninterrupted for a further forty-six hours at the end of which period the flux was still over 130 gfd (5291 ℓmd). Thus, this specific cleaning procedure appears to work quite successfully for pink water wastes.

In addition to single-tube tests, studies were conducted with a small module containing seven 9 mm tubes, for a total filtration surface area of 2.5 ft^2. Initial flux was ~ 280 gfd (Fig. 18). After 165 hours of uninterrupted operation with the module, the flux was still about 70 gfd (2800 ℓmd). Note that in the type of long-term tests being described herein, the same waste feed is repeatedly recirculated through the modules many times.

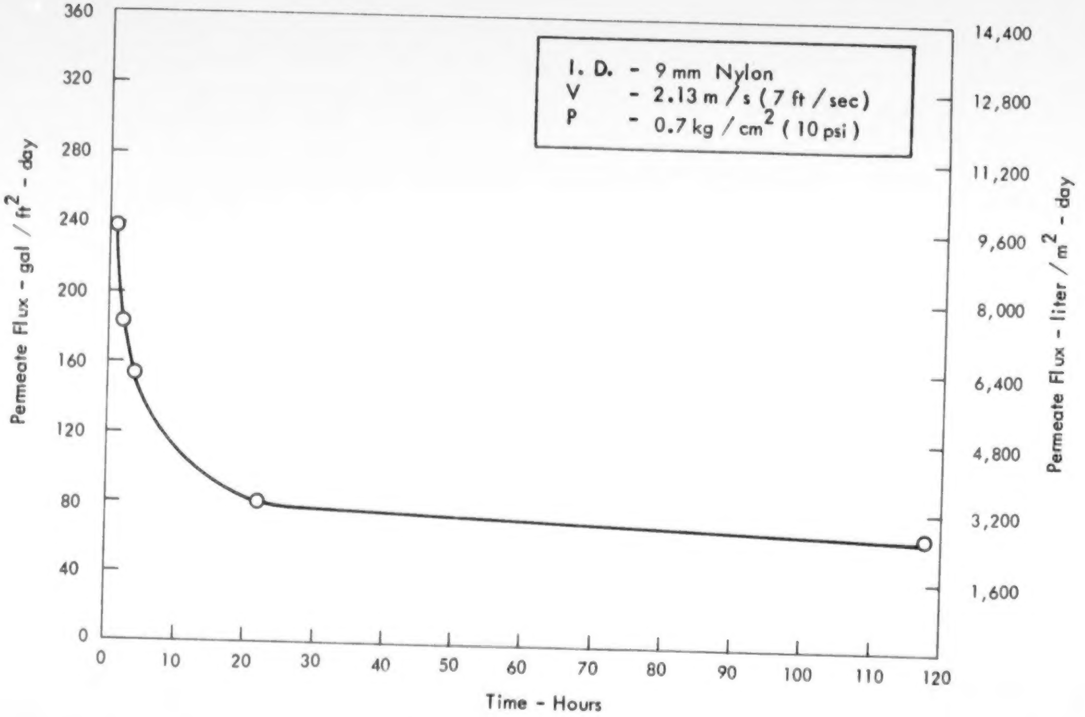


Figure 16 - Pink Water Tests with 9 mm Nylon Tube

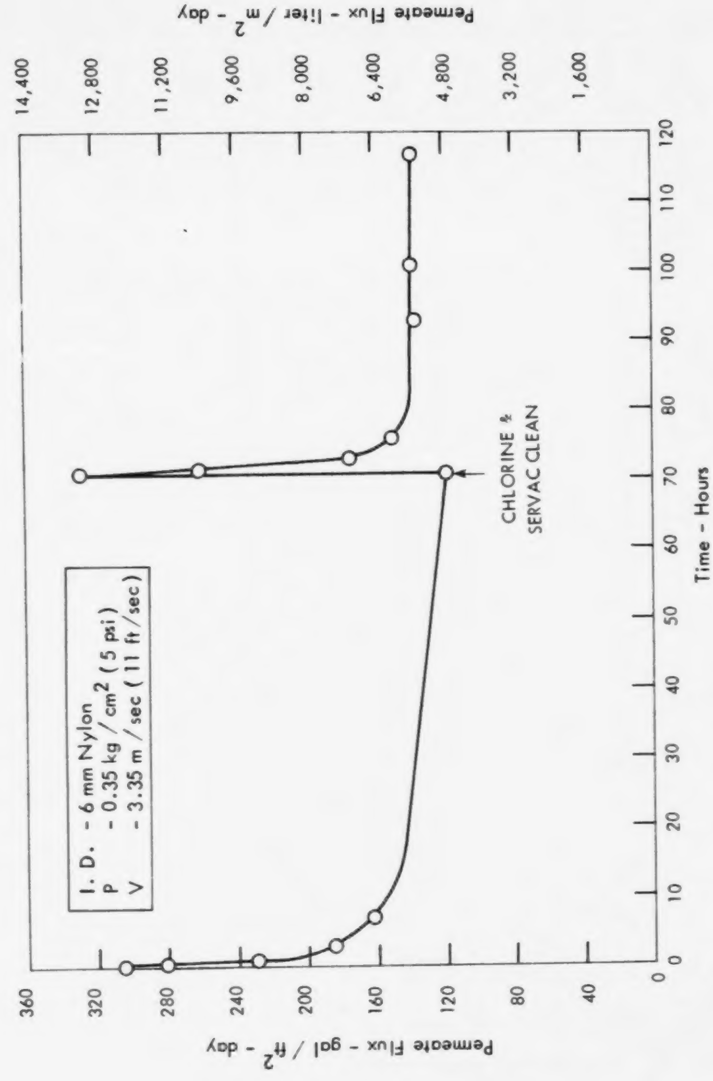


Figure 17 - Pink Water Test with 6 mm Nylon Tube

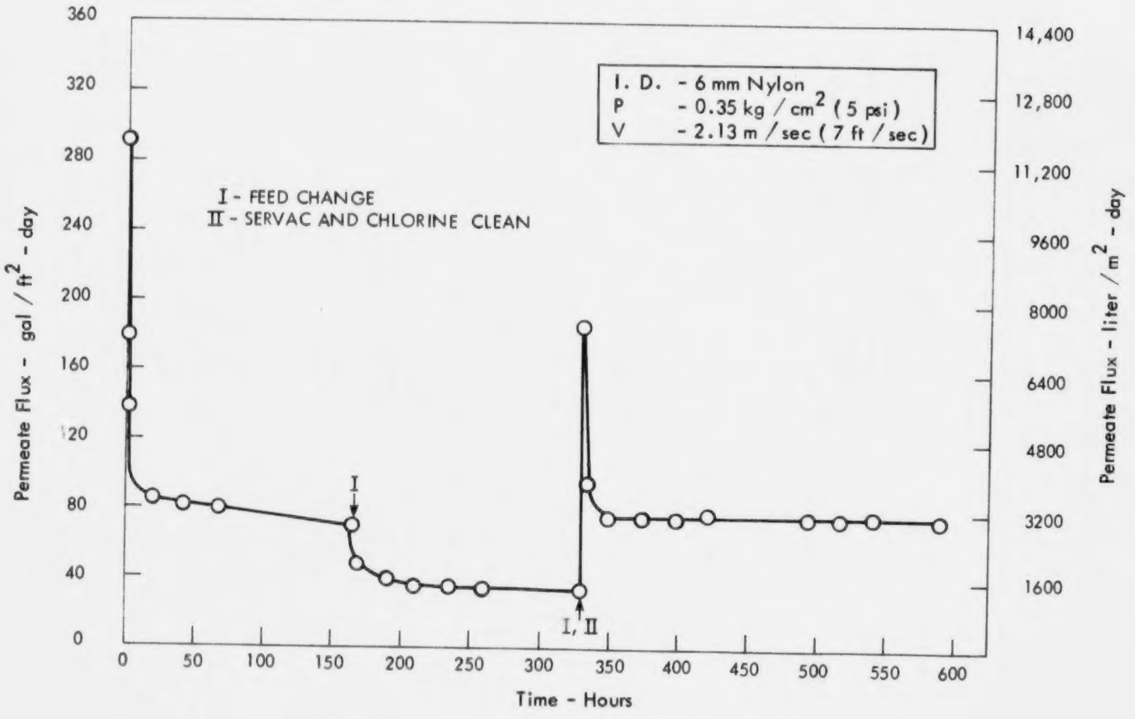


Figure 18 - Pink Water Module Test for Long-Term Flux Behavior

Such repeated recirculations through the pump may lead to some changes in the characteristics of the circulating feed and thus yield nonrepresentative results. To examine this issue, the circulating feed was replaced by a fresh batch after 165 hours of operation, but the tubes were not cleaned. When the test was restarted, the observed flux decline was indeed larger, although within a few hours a new "plateau" value was reached, equal to about one-half of that before the feed change.

After a total of 330 hours of operation, the flux was still almost 40 gfd. At this point the module was cleaned successively with Servac and hypochlorite solutions, utilizing the procedure described previously. After cleaning, the test was restarted with a fresh batch of pink water wastes. As can be seen from Fig. 18, cleaning increased the flux to about 185 gfd (7540 ℓmd) although within about fifteen hours the flux had declined to 78 gfd (3200 ℓmd). During the next three-hundred-and-fifty hours of operation there was little or no change in the flux!

PINK WATER PERMEATE QUALITY

Analyses of the pink water feed showed total solids ~ 380 ppm, SS ~ 81 ppm, and TOC ~ 92 ppm. Treatment with Hydroperm achieved virtually complete removal of SS (< 5 ppm), with reduction of TS to < 200 ppm and TOC to ~ 65 ppm.

The quality of the permeate thus appears to be fully adequate for reuse of pink water in demilitarization operations. In addition, the virtually complete removal of SS will substantially prolong carbon column life where this polishing step is needed for treatment for discharge.

SECTION VI - THE HYDROPERM™ SYSTEM

The basic element of the Hydroperm cross-flow microfiltration system is, of course, the tubular filter element which was described in Section II. The design of the total system first involves assembling large numbers of these tubes into modular form in a manner which is economical in terms of capital and operating costs. These modules make up the basic building blocks of any system.

A number of steps are required in total system design, starting with tube optimization and proceeding to component selection and sizing. The modules can be optimized in terms of their length and diameter, number of tubes contained, type of end-fitting used, and so on. A typical module is shown in Fig. 19. The arrangement of the modules can also be optimized in terms of whether they are arranged in series, in parallel, or in a combination. The criteria used for the last two optimization steps are ease of handling, ease of maintenance, ease of installation, power requirements, space constraints, as well as capital and operating costs.

The modules themselves make up only one part of the "total" system, which includes pumps, valves, gauges, feed holding tank, sludge-thickening tank, permeate holding tank, module cleaning system, etc. A typical schematic of

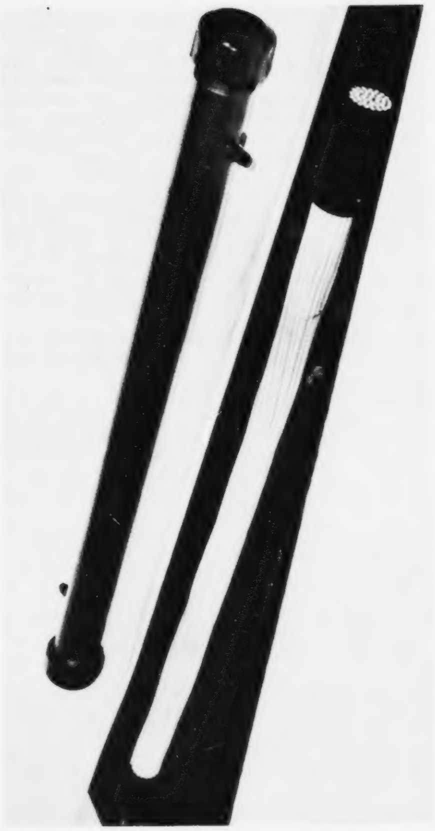


Figure 19 - HYDROPERM™ Module

a Hydroperm wastewater treatment system is shown in Fig. 20.

SECTION VII - CONCLUSIONS

In this paper results are presented from laboratory testing of the Hydroperm cross-flow microfiltration system with the U. S. Army's "MUST" hospital wastewater, "pink water" from Army Ammunition Plants, and wastewater from domestic and industrial laundries. In all cases, the tests were designed to establish the feasibility of producing reusable water. The results of the tests showed that the Hydroperm system achieved virtually complete removal of suspended solids (the filtrate water had a suspended solids concentration of < 5 ppm) regardless of the feed concentration. Also substantial removal of dissolved solids, BOD and COD was demonstrated. The duration of various tests ranged from days to weeks, and it was shown that flux levels of tens to hundreds of gallons/ft^2-day, depending on the waste, could be achieved for long periods of time with simple ten-minute cleaning operations at daily intervals. Both from a flux and a filtrate quality standpoint, the tests indicate that the Hydroperm system is suitable for producing reusable water for these three applications.

When compared with other systems, Hydroperm offers several unique advantages. These are summarized below:

(a) Compactness: Hydroperm filtration systems, unlike biological and chemical systems, do not require large areas. Indeed, they can be engineered to fit available space. Because of its modular construction, the Hydroperm system can be designed to accommodate a full range of wastewater treatment requirements, from small to large.

(b) Ruggedness: Since they are made from inert thermoplastics, the performance of Hydroperm tubes does not depend, in general, on changes in influent pH. Moreover, due to their rugged structure and low operating pressure, Hydroperm modules are not subject to the fouling and leaking problems which have plagued some membrane systems; nor are they subject to clogging in the presence of oily wastes.

(c) Ease of Maintenance: Because of their ruggedness and modular construction, Hydroperm systems are easy to maintain. They can be engineered in such a way that a failure in a given module causes only a small part of the total system to be shut down.

(d) Product Recovery: In many cases product recovery is possible.

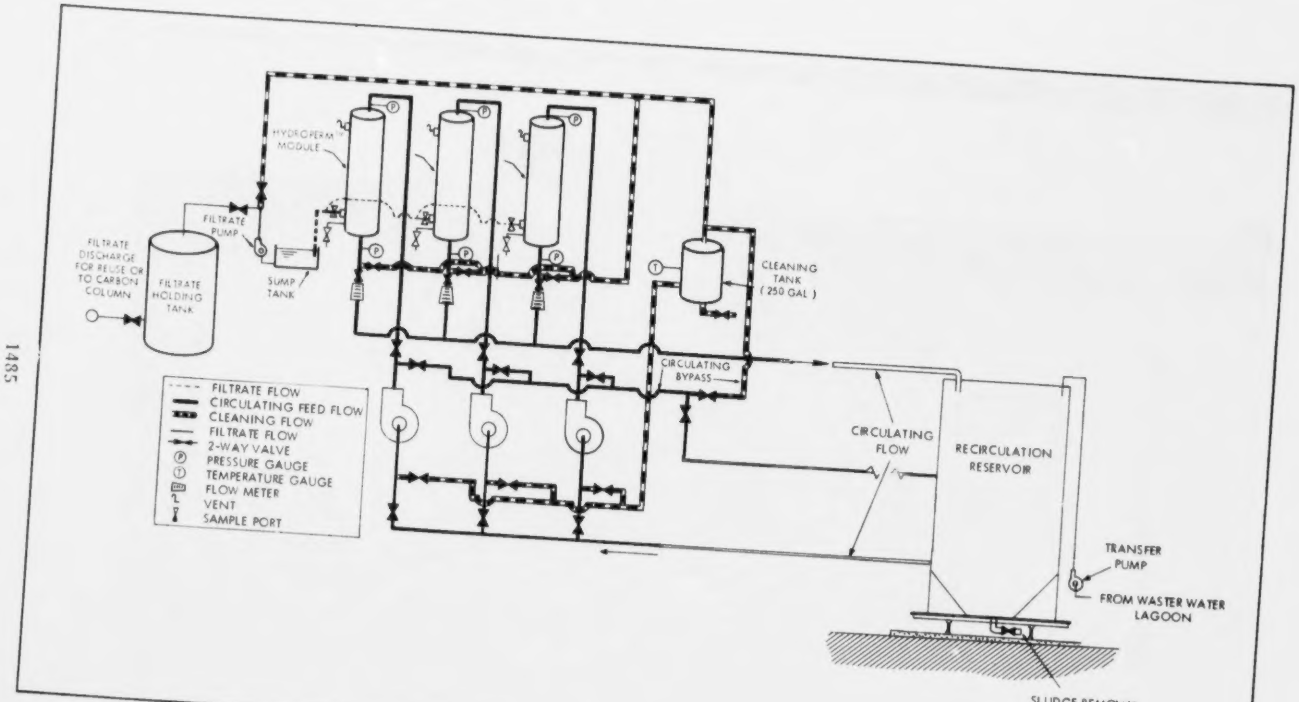
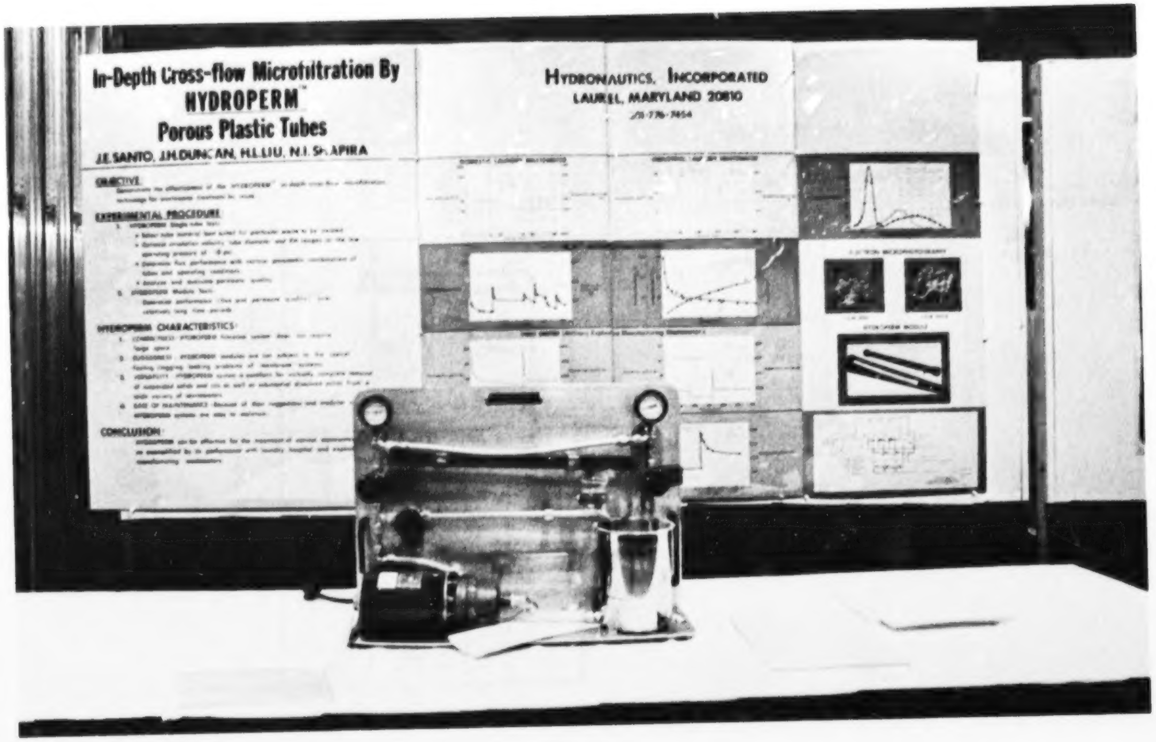


Figure 20 Schematic of HYDROPERM™ Wastewater Treatment System



In-Depth Cross-flow Microfiltration By HYDROPERM™ Porous Plastic Tubes

J.E. SANTO, J.H. DUNCAN, H.L. LIU, N.I. SHAPIRA

HYDRONAUTICS, INCORPORATED
LAUREL, MARYLAND 20810
301-776-7454

OBJECTIVE:

EXPERIMENTAL PROCEDURE:

HYDROPERM CHARACTERISTICS:

CONCLUSION:

REDUCTIVE DEGRADATION TREATMENT OF INDUSTRIAL AND MUNICIPAL WASTEWATERS

Keith H. Sweeny
Manager, Wastewater Treatment Research
Envirogenics Systems Company
El Monte, California 91731

An important problem facing those concerned with maintaining clean waters is from the discharge of halogenated hydrocarbons into the waters. These discharges may arise from manufacturing and processing a variety of chemicals, and upon chlorination of some municipal waters. An area of particular concern is the treatment of the chlorinated hydrocarbons soluble in water, since the soluble material can not be removed by filtration, and is generally resistant to conventional biological treatment.

An economical and effective means for the treatment of the dissolved halogenated toxicants would permit the reuse of these waters in manufacturing or processing plant recycle, allow the waters to be used safely for ground water recharge, and to permit safe discharge into rivers and other waters. Treatment of the reportedly carcinogenic trihalomethanes (THM's) produced by the chlorination of water containing organic matter is another important application leading to better use of our waters.

A new and novel approach to the treatment of these waters is the technique of reductive degradation. This process appears to be highly effective for the treatment of halogenated toxicants, is very simple, and is projected to be an economical treatment process. In this paper the process will be briefly described, some observed reactions of degradation given, and some materials known to be degraded will be listed. The application of the process to the destruction of trihalomethanes will then be described and a projection of the treatment cost to these THMs will be given.

PROCESS

The process is basically one of chemical reduction of the contaminant by a catalyzed metal powder. The reaction can be carried out at ambient temperatures, though the reaction is expectedly more rapid at slightly elevated temperatures.

Reductants

The reductants generally employed have been the powders of iron, zinc and aluminum. All of these metals are effective reductants, though iron is generally preferred on the basis of effectiveness at low cost and freedom from deleterious reductant products; zinc ion in the water, for example, would offer toxicity problems to aquatic life unless removed.

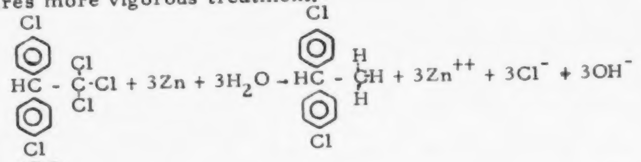
Process Chemistry

The reduction reactions are known to give several different products, depending on the material to be reduced, the reductant, and reduction conditions. Some of the known degradation reactions can be illustrated.

Hydrogenolysis. The most common of the reductive reactions appears to be the replacement of a halogen atom by a hydrogen.

$$R\,Cl + H_2O + Fe \rightarrow RH + Fe^{++} + OH^- + Cl^- \qquad (1)$$

Removal of the halogen atom generally leads to products of greatly reduced toxicity. The initial reduction of DDT by zinc, for example, proceeds by reaction (1). Further removal of the aromatic chlorines requires more vigorous treatment.



DDT

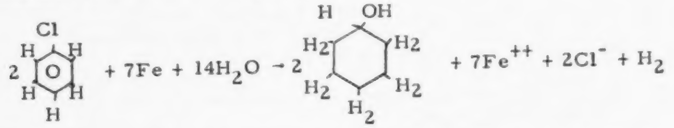
Hydroxylation. Substitution of a hydroxyl group for the halogen has also been observed.

$$2RCl + Fe + 2H_2O \rightarrow 2ROH + Fe^{++} + 2Cl^- + H_2 \qquad (2)$$

Saturation. Saturation of an aromatic structure has likewise been shown in tests involving the reduction of halogenated organic compounds.

$$RCH = CHR + Fe + 2H_2O \rightarrow RCH_2CH_2R + Fe^{++} + 2OH^- \qquad (3)$$

An example of both hydroxylation and saturation is shown in the case of the catalyzed iron reduction of chlorobenzene, where cyclohexanol has been observed as a reaction product.

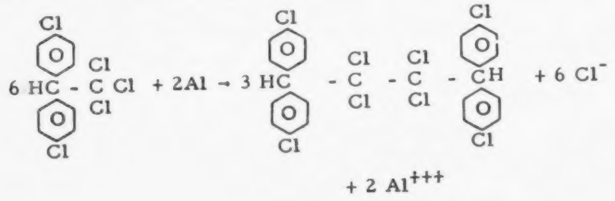


In a related test, reduction of hexachlorocyclopentadiene led to the production of non-chlorinated branched chain alkanes - evidence of dechlorination and ring opening.

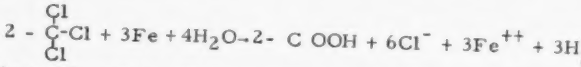
Condensation. A reductive condensation has also been found to occur when certain toxicants are treated with the reducing agents.

$$2RCl + Fe \rightarrow R - R + Fe^{++} + 2Cl^- \qquad (4)$$

An example of this reaction is shown with the reduction of DDT by catalyzed aluminum to produce the butane derivitive with extremely low water and fat solubility.



Hydrolysis. Hydrolysis has also been observed in the presence of the reductants. While the hydrolysis reaction is not considered to be a reduction, the hydrolytic reaction has been observed in the presence of the catalyzed metal reductant, and was not observed in its absence. For example, in the presence of catalyzed iron, the aliphatic chlorines in DDT are apparently hydrolyzed to give the acid (as well as the tetraphenyl tetrachlorobutane).



$$2 - \overset{Cl}{\underset{Cl}{C}}\text{-}Cl + 3Fe + 4H_2O \rightarrow 2 - C\,OOH + 6Cl^- + 3Fe^{++} + 3H_2$$

Reaction with water. The amount of iron consumed in the reductive reaction is generally somewhat greater than the calculated amount consumed by one of the preceeding reactions. The additional iron is apparently consumed by reaction of the iron with water:

$$Fe + 2H_2O \rightarrow Fe^{++} + 2OH^- + H_2 \qquad (5)$$

Typically, the effluent from the reductive column will contain one to three mg/ℓ of ferrous iron when an industrial or municipal water is fed to the unit, and less than 1 mg/ℓ when deionized water is employed. This amount of iron can be readily removed, if desired, by standard aeration and flocculation methods.

Materials Reduced

Tests with a number of chlorinated and nonchlorinated species shows that the reductive technique has wide applicability. Some of the variety of materials reduced are shown in Table 1. The effectiveness of degradation of several types of materials are shown in Table 2. These materials include the important cyclodiene pesticide endrin, hexachlorocyclopentadiene and the pesticide Kepone produced from it. Toxaphene, a widely used chlorinated camphene, and atrazine, a triazine derivitive represent two of the most widely used organic pesticides. Two non-chlorinated species from the "consent-decree" list of chemicals, the reportedly carcinogenic N-nitrosodimethylamine (or dimethylnitrosamine), and p-nitrophenol are also shown. These illustrations have been selected to show the wide range of chemicals known to be reductively degraded by the Envirogenics process. In all of these examples, complete or near complete degradation is obtained, and the analysis of the waters shows that the bulk of the chlorine is removed from the organochlorine compounds. The chemical changes that appear to have taken place in these species all lead to substantially reduced toxicity so that the effluent would appear to be innocuous at the levels expected to be encountered.

PROCESS OPERATION

While many organic compounds can be degraded by mixing the reductant powder in a solution of the organic compound, much better degradation has been achieved by use of a column. Two versions of the column have been employed, the diluted bed and the fluidized bed.

Reduction is best accomplished by using a large excess of the reductant and a contact time up to 100 minutes at ambient (~ 25°C) temperature. These conditions can be achieved with a bed of the reductant diluted with an inert ingredient (such as sand) to give requisite flow and retention properties. The sand-diluted bed may contain 5 to 40 wt-% of the reductant and the flow through a 6 ft. deep bed may range from 0.5 to 3 gpm/sq ft. In order to provide suitable retention and flow properties, sand with approximately the same particle size as the reductant powder is used, so that interstitial particle packing is minimized. Substantial differences have been found in sand diluents; some sand sources retard the rate or extent of degradation. A nominal 100 gpm reductive treatment unit using a sand-diluted bed is shown in Figure 1.

A fluidized bed reactor is even more efficient. In this process, the catalyzed reductant powder in a column is supported on a distributor plate such as a fine mesh sieve or a porous metal plate. Flow of the liquid into the bottom of the vertical bed expands the powder mass, giving turbulent motion to provide good mixing of the reductant and wastewater to be treated. In a typical reactor, the bed may be expanded two-to-four-fold from the depth of the quiet bed. Flow rates as high as 22 gpm/sq ft have been achieved with good degradation - this flow rate is approximately ten-fold greater than for the sand-diluted bed. The pressure drop across the system is low, with current systems operating at approximately 15 psi. Contact times are generally of the order of 5 min.

While wastewater streams treated with a sand-diluted reductant bed must have adequate filtration before the reactor, filtration does not appear to be required with the fluidized bed unit. Particulate matter entering a sand-diluted reactor gradually clogs the bed, and any attempt at backwashing leads to particle segregation, since the reductant particles are about three times as dense as the diluent particles. In the fluidized bed, however, a moderate amount of particulate matter of about the same size as the reductant is expected to pass through the reactor into the overflow, because of lower density. Thus, the fluidized bed reactor gives promise of direct reaction of chlorinated organics in silt, in which the decontaminated silt would pass through the bed into the discharge.

DEGRADATION OF "TRIHALOMETHANES"

The degradation of the "trihalomethanes" has been shown to be effectively achieved in a series of tests.

The initial tests were made with sand-diluted beds in which the toxicant-laden water was allowed to trickle through the 6-ft deep bed by gravity flow. Flows ranging from 30 to 45 mℓ/min through a 37 mm dia. bed were employed in the initial test. Samples containing 100 μg/ℓ each of chloroform, carbon tetrachloride, methylene chloride, and bromochloromethane were prepared and passed through the column. Samples of the effluent analyzed by gas chromatography (electron-capture detection) showed no trace of the starting material (< 1 μg/ℓ). Later tests were made with a fluidized bed reactor, employing a laboratory unit in which a flow as high as 1ℓ/min was passed through the 37 mm dia. column. This test was carried out with a 100 μg/ℓ aqueous chloroform solution and the effluent was reduced to detection limits (< 1 μg/ℓ) after passage through the reductive column.

Bench-Scale Degradation of Chloroform

Following these tests, an apparatus was constructed for a larger-scale test on the degradation of aqueous chloroform. A fluidized bed reactor was constructed from 6 in. dia. glass pipe; the column was 9 ft high. A distributor plate of fine sieve screen was used for flow distribution and to support the reductant bed. Industrial water (no detectable chloroform) was passed into the column, and chloroform solution (in methanol) was injected into the stream to give a 100 μg/ℓ level of the toxicant in the water. The bed was operated for a period of about 3 months with a total flow of 166,000 gal. through the apparatus. Flow rates ranged as high as 3 gpm (~ 16 gpm/sq ft). Samples were collected, extracted with methylcyclohexane by the standard EPA liquid-liquid extraction method (Reference 1) and analyzed by gas chromatography (electron-capture detection). None of the effluent samples was found to contain measurable amounts of chloroform.

Laboratory Tests of Bromoform Degradation

The degradation of aqueous bromoform has also been demonstrated. An aqueous solution containing 250 $\mu g/\ell$ of bromoform was passed through a laboratory fluidized bed column (37 mm dia.) at 390 to 450 $m\ell/min$ (equivalent to 8.9 to 10.3 gpm/sq ft). Analysis of effluent samples after 10, 20, and 30 ℓ flow showed no detectable amount (<0.1 $\mu g/\ell$) of either bromoform starting material or dibromomethane as a potential product. The analyses were also made by the EPA liquid-liquid extraction method and measured by electron-capture detection gas chromatography (Reference 1).

TREATMENT COST

The simplicity of the reductive degradation process results in low estimated operating and capital costs. A schematic of the general process is shown in Figure 2.

The reduction column operates most satisfactorily at a pH near 7; too low a pH results in excessive reductant consumption, and too high a pH leads to reductant precipitation in the bed, as well as decreased efficiency. If the pH is less than 5.5 or greater than 8.0, pH sensing and neutralization are necessary. However, with the fluidized bed system and a nearly neutral stream, the system may be as simple as a pump, flow controls, and the reductant bed, as is shown in Figure 2.

An estimate of the treatment cost for a fluidized bed system, using the method and the data from a recent study on granular activated carbon absorption (Reference 2) was made. The results of this estimate are compared with those of the referenced study in Table 3. These data indicate substantially lower projected treatment costs for the reductive technique as compared to granular activated carbon treatment. These calculations were made for a municipal water source that did not require pH neutralization, filtration, or iron removal.

CONCLUSIONS

1. Reductive degradation treatment has been shown effective for a wide range of halogenated toxicants dissolved in water. The technique has also been shown to be useful for the non-chlorinated species investigated.

2. Treatment costs are estimated to be significantly lower for reductive degradation than for activated carbon, when compared for trihalomethane treatment at a variety of water plant sizes. Capital costs are also estimated to be much less for reductive treatment.

3. Products that have been identified, as well as expected products from the known reactions, all lead to apparently safe products.

4. The process is inherently very simple, requiring minimum labor and space.

ACKNOWLEDGMENT

Some of the early portions of this work were supported by the U. S. Environmental Protection Agency, whose support was appreciated.

REFERENCES

1. The Analysis of Trihalomethanes in Drinking Water by Liquid/Liquid Extraction. U. S. EPA, Environmental Monitoring and Support Laboratory, Cincinnati, Ohio. (Sept. 9, 1977).

2. Appraisal of Powdered Activated Carbon Processes for Municipal Wastewater Treatment. U. S. EPA Report 600/2-77-156. Cincinnati, Ohio (Sept. 1977) Tables 67, 68, 69, 70, 71.

TABLE 1

Organic Chemicals Reduced

Chlorinated

Cyclodienes
  aldrin
  chlordane
  dieldrin
  endrin
  heptachlor
  heptachlor epoxide

Chlorinated Camphene
  toxaphene

Light Ends
  carbon tetrachloride
  chloroform
  methylene chloride
  chlorobromomethane

s-Triazine
  Atrazine

DDT-Type
  DDT
  DDD
  Kelthane
  methoxychlor
  Perthane

Chlorinated Cyclohexane
  lindane

Chlorinated Phenoxyacetic Acid
  2, 4-D ester
  2, 4, 5-T ester

Hexachlorcyclopentadiene Derivatives
  hexachlorcyclopentadiene
  Kepone

Non-Chlorinated

p-nitrophenol
N-nitrosodimethylamine

TABLE 2

Effectiveness of Degradation

Toxicant	Influent	Effluent	Principal Products
Endrin	100 µg/ℓ	<0.05 µg/ℓ	non-chlorinated non-aromatic
Hexachlorocyclopentadiene	30 µg/ℓ	<0.1 µg/ℓ	non-chlorinated branched chain alkane
Kepone	2.0 mg/ℓ	0.5 µg/ℓ	
Toxaphene	1.0 mg/ℓ	not detected	
Atrazine	60 mg/ℓ	0.6 mg/ℓ	
p-nitrophenol	1.0 mg/ℓ	1.5 µg/ℓ	no phenol or aniline
N-nitrosodimethyl amine	1.0 mg/ℓ	none detected	no dimethylamine

TABLE 3

Estimated Treatment Costs for Reductive Degradation Treatment of Municipal Waters and Wastewaters

	$/MGD*				
	Flow, MGD				
	1	5	10	25	50
Operating Costs, 1 Pass					
Power & Supplies	37.30	35.65	35.67	33.75	33.74
Labor & Maintenance	69.97	22.64	17.75	13.91	11.79
Total Operating Cost	107.27	58.29	53.42	47.66	45.53
Amortized Capital (7%, 20 yr)**	30.92	25.94	18.67	18.56	18.40
TOTAL TREATMENT COSTS	138.19	84.23	72.09	66.22	63.93
$/1000 gal	0.138	0.084	0.072	0.066	0.064
Operating Costs, 2 Pass					
Power & Supplies	70.52	68.03	67.51	64.38	64.38
Labor & Maintenance	83.20	32.15	24.88	22.77	20.42
Total Operating Cost	153.72	100.18	92.39	87.15	84.80
Amortized Capital	55.95	47.36	34.09	35.70	35.34
TOTAL TREATMENT COST	209.67	147.54	126.48	122.85	120.14
$/1000 gal	0.210	0.148	0.126	0.123	0.120
Treatment Costs for Activated Carbon (Ref. 2) $/1000 gal	1.84	0.73	0.58	0.46	0.40
Estimated Capital Costs,	1000$				
for Reductive Treatment					
1 pass	58	245	352	875	1,736
2 pass	106	447	643	1,683	3,333
for Carbon Treatment, (Ref. 2)	4,737	8,325	12,667	22,861	38,840

---

* MGD = Million gallons per day, 24 hr/day, 7 day/week operation.

** 7% was used in the referenced study and was used here for comparative purposes.

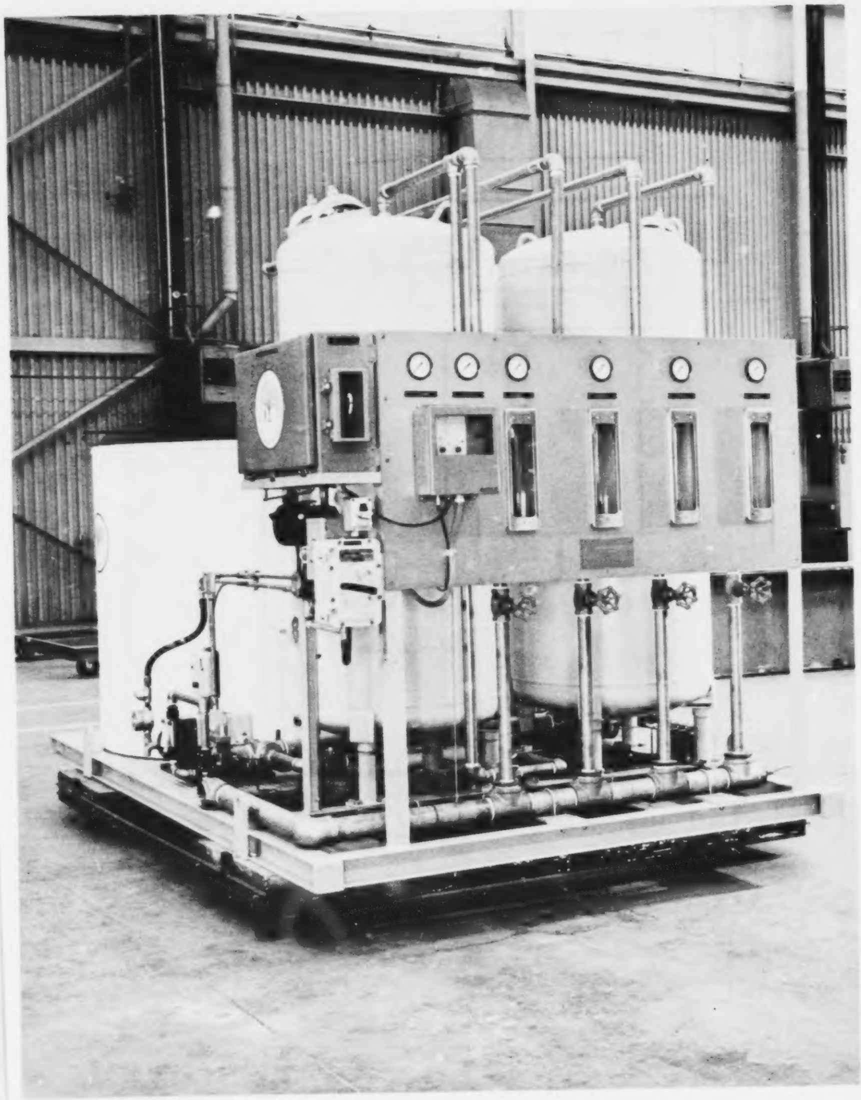
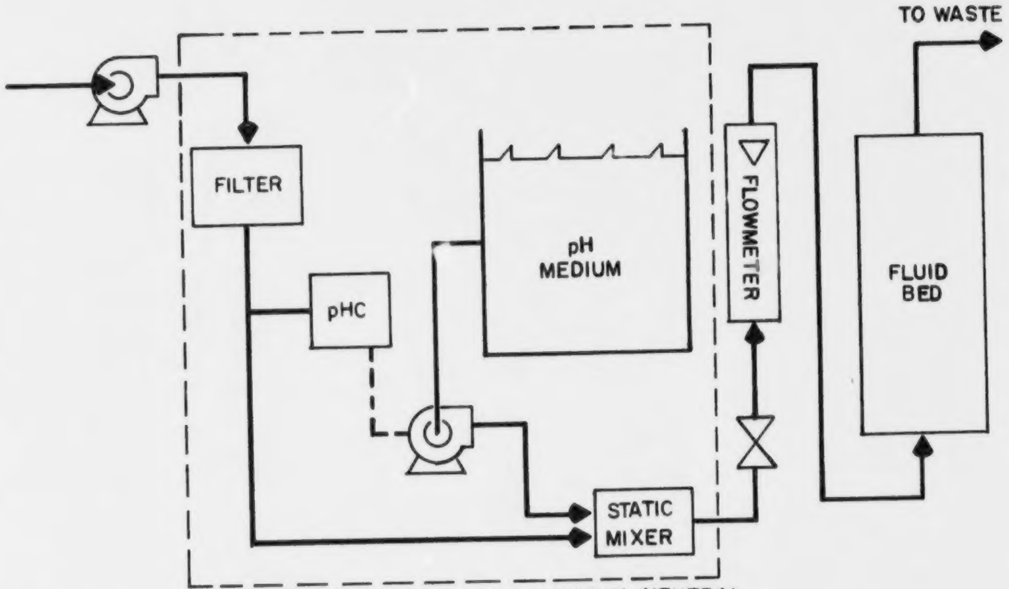


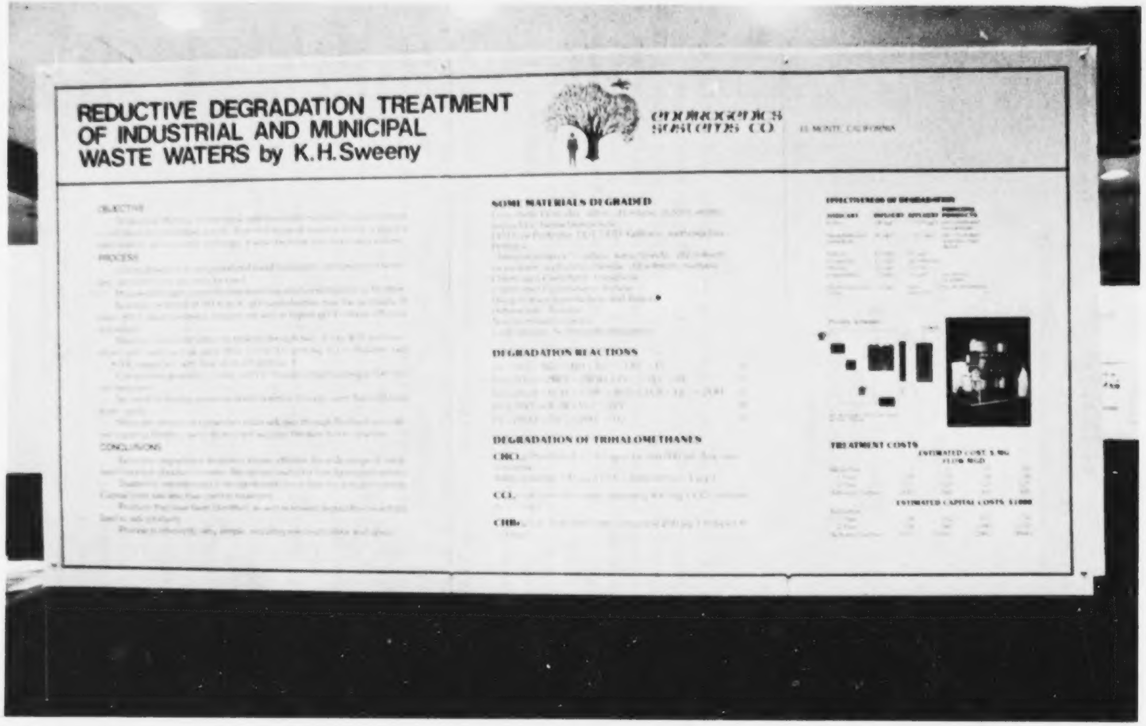
FIGURE 1

100 GPM Sand-Diluted Reductive
Treatment Plant



NOT NECESSARY FOR CLEAR, NEARLY NEUTRAL STREAM.

FIGURE 2 REDUCTIVE TREATMENT SYSTEM



REDUCTIVE DEGRADATION TREATMENT
OF INDUSTRIAL AND MUNICIPAL
WASTE WATERS by K.H.Sweeny

ENVIROGENICS SYSTEMS CO.
EL MONTE, CALIFORNIA

OBJECTIVE

PROCESS

CONCLUSIONS

SOME MATERIALS DEGRADED

DEGRADATION REACTIONS

DEGRADATION OF TRIHALOMETHANES

$CHCl_3$

CCl_4

$CHBr_3$

EFFECTIVENESS OF DEGRADATION

TREATMENT COSTS

ESTIMATED COST $/MG
FLOW MGD

ESTIMATED CAPITAL COSTS $1000

SODA ASH TREATMENT OF ACID MINE DRAINAGE IMPREGNATED STREAMS FOR POTABLE WATER USE

David A. Long
Associate Professor of Civil Engineering
The Pennsylvania State University
University Park, PA

James A. Butler
Engineer
Gwin, Dobson & Foreman, Inc.
Altoona, PA

Michael V. Lenkevich
Chemist
Gwin, Dobson & Foreman, Inc.
Altoona, PA

Introduction

Lime neutralization of coal mine drainage is a well known and widely aplied process for upgrading stream water quality. When a significant portion of the watershed available to a municipality for water supply is affected by mine drainage pollutants, the question as to whether or not neutralized mine drainage can be economically treated to a more beneficial use level becomes a logical consideration. The $4.8 million water treatment plant in Altoona, located in south-central Pennsylvania, was constructed with "Operation Scarlift" funds ($500 million Pennsylvania bond issue money) with a view to addressing both of these problems, i.e., stream pollution abatement and municipal water supply.

The watershed involved drains a portion of the mountainous area to the southwest of Altoona in the vicinity of Penn Central Railroad's well-known Horseshoe Curve. The after effects of deep mining coal in this region has rendered two streams acid, --Glen White Run and Kittanning Run. The latter, with only half the drainage area of the former, is about seven times more acid than Glen White Run. Early in Altoona's history, a series of three raw water reservoirs, together with transmission lines, were constructed to provide a water supply for the City during periods when the high quality source (Mill Run Reservoir) became depleted. Under normal conditions, the backup reservoir system in the Horseshoe Curve consisted primarily of the better source, Glen White, with Kittanning Run bypassing the impoundments altogether. During severe drought, however, there were times when the highest reservoir (Kittanning Point) was fed by the Kittanning Run source, and this poor quality of water was the only one available to City consumers.

The rationale for the new treatment plant was to provide two levels of treatment: 1) lime neutralization of Kittanning Run to meet stream release standards (pH 6.0 to 9.0 and iron 7.0 mg/1), and 2) lime neutralization and lime/soda ash softening of the reservoir system supply, fed by the Glen White source, to meet EPA potable water standards. The study was prompted by the question, would it be technically and economically feasible to treat the Kittanning Run raw water source to a more beneficial-use quality? It was the main objective of this project to develop the basis for considering this question within the limits imposed by the available treatment units.

Process Description

The treatment system available for study under this project is shown diagrammatically in Figure 1. The operational units employed are grouped together according to the major process objectives, neutralization, softening and sludge handling. The first stage, or neutralization process, consists of the flash mix, slow mix, aeration and sedimentation units. Detention times in this flow-through sequence are 25 seconds, 25 min, 1.1 hr, and 6.17 hr respectively based on a flow of 13,200 cu m/day (3.5 MGD). Settled solids are pumped from the sedimentation basin at a controlled rate to the flash mix (recycled sludge) and to the thickener clarifier (waste solids). Lime slurry is fed to the flash mix in a controlled amount to maintain the pH called for in the slow mix tank. In addition, as a result of this investigation, provision has been made to feed potassium permanganate into the influent raw water line.

Effluent from the first stage sedimentation basin meets stream release water quality and, in the case of Kittanning Run, would normally be discharged from the plant. For purposes of this study, however, the effluent was directed to the mix box where lime can be added prior to softening and recarbonation. The softener unit is a circular tank arranged with separate compartments to provide zones for rapid mix, flocculation, and clarification (Walker Process ClariFlow Unit). In theory, the unit is designed to pump settled solids from the bottom-center of the tank back to the reaction zone. Chemicals (soda ash, alum, coagulant aid, and potassium permanganate) can also be fed at this point to allow for mixing of the applied flow with previously formed floc and chemicals so that the required reactions, coagulation, and flocculation can occur. Theoretical residence times for the reactor and settling zones are 25 minutes and 83 minutes respectively. Flow from the softener is to the recarbonation chamber 12.2m x 2.4m x 3.0m (40' x 8' x 10') where carbon dioxide is added. Recarbonation detention time for the 13,200 cu m/day (3.5 MGD) flow is 10 minutes. The effluent from the recarbonation unit is filtered prior to clearwell storage and chlorination.

The major components of the sludge handling process are the thickener clarifier, holding tank and vacuum filters. For purposes of this discussion the backwash lagoons, which receive backwash water from the filters, filtrate from the vacuum filter, and thickener overflow, also are included in the solids handling operation. The thickener receives underflow solids from the first stage (sedimentation basin) and the second stage (softener). Thickened sludge is pumped to a holding tank and from there to the vacuum filters as required. The vacuum filters are elevated so that the sludge cake can be conveyed to a hopper and periodically loaded on a truck for hauling to final disposal.

Process Engineering and Operation

Project Plan. The total project was divided into several major phases. The first phase was designed to achieve an effluent quality which would meet EPA requirements for potable water. At the time the study was conducted (August 1974 - February 1975) the Interim Primary Drinking Water Standards required under the Safe Drinking Water Act (1) had not been established. Therefore, the 1962 Public Health Service Drinking Water Standards (2), except for sulfate content, were used to define desired effluent quality. Specific objectives were to produce a finished water with approximately 100 mg/l $CaCO_3$ hardness and containing no more than the permissible 0.3 mg/l and .05 mg/l of iron and manganese respectively. This product water was designated as "municipal water supply quality" (MWSQ).

During each phase of the study, operational parameters were varied in an attempt to determine the optimum operating conditions to produce the desired effluent quality for that phase of the study. The primary parameters which were varied were pH, amount of chemicals added, and the point of chemical application. Various pH's were set at different stages in the overall treatment process to see which scheme would minimize operating problems while producing an acceptable effluent quality. To clarify this further, during parts of the study the desired process pH was achieved initially in the neutralization stage and then was carried right on through the softening stage while in other parts the pH's were varied in the first stage unit as a pre-treatment (iron and manganese oxidation stage) and the pH's for the softening reactions were regulated in the reactor-clarifier unit.

While it would have been highly desirable to have been able to obtain filter operating data for the duration of the project, it was not possible to do so under the operating conditions encountered. Therefore, the majority of the data analysis presented will be in terms of the softener and recarbonation basin effluent characteristics only. Filtration of the recarbonation basin effluent would undoubtedly yield effluents with somewhat different characteristics than those reported herein but it was impossible to gather such data during this study other than for a limited period during the second phase. Generally, filtration would be expected to produce water with lower hardness, iron, and manganese than reported due to "seasoning" of the filters. Only the results of the portions of the study concerned with producing MWSQ water will be presented herein.

Raw Water Quality. Raw water for the study was a blend of Kittanning Run (K.R.), a moderate acid mine drainage affected source, and Impounding Dam (I.D.), a better quality water. Design flow through the portion of the plant used for the study was 13,200 cu m/day (3.5 MGD). Because the sustained yield of Kittanning Run is less than 13,200 cu m/day (3.5 MGD), it was necessary to blend the two sources as indicated in order to achieve the design flow. Table 1 shows the blended water characteristics for a blend typical of that treated during the study, i.e., 5680 cu m/day (1.5 MGD) of K.R. plus 7570 cu m/day (2.0 MGD) of I.D.

Analytical Methods. Samples for the various analyses performed during the study were collected daily and composited by treatment plant personnel. Following collection, the samples were transported to the laboratory for analysis.

EPA (3) methods were used for the determination of acidity, alkalinity, suspended and dissolved solids, aluminum, calcium, iron, magnesium, manganese, sodium, and sulfate.

"Standard Methods "(4) procedures were used to determine pH and specific conductance. All hardness values reported herein were calculated in accordance with "Standard Methods" using the values obtained for Ca, Mg, Al, Mn, and Fe.

Data Presentation. The nature of this study was such that a vast amount of data was collected under varying operating conditions. Also, because the study was conducted at full plant scale, process control was more erratic than would be expected if conducted at laboratory or pilot scale. The data, therefore, showed considerable variability. All operating data obtained during the course of the study were included in the data analysis unless there was an obvious error in sampling or laboratory analysis. It was felt that data could be presented best in the text of the report by means of probability plots that show the statistical correlation of the data.

Data analysis for the preparation of the probability plots was done in accordance with the procedures outlined in the Handbook for Monitoring Industrial Wastewater (5). Data which are normally distributed yield a straight line when plotted on probability paper. In some instances, abnormal breaks in the data are observed on such plots. This observation indicates that some change in operating conditions occurred which resulted in data being obtained from a different distribution and hence, they do not plot along the original straight line. These data may, however, plot on a different straight line showing a normal distribution of data obtained under the changed operating conditions.

Municipal Water Supply Quality Results. For the purposes of this project, municipal water supply quality was defined as treated water having a total dissolved solids (TDS) content of less than 500 mg/l. Further, the target hardness for the treated water during this portion of the study was 100 mg $CaCO_3$/l. Also, as indicated earlier, effluent limits for iron and manganese were 0.3 mg/l and 0.05 mg/l, respectively.

During this phase of the study, data were collected over a total of 74 days which were divided into 11 different operating periods. Process control conditions for these periods are summarized in Table 2. Because of operating problems with the softening unit, which will be discussed in more detail later, it was deemed advisable to vary pH's and other operating parameters as shown in an attempt to improve performance of the softening unit.

Examination of the characteristics of the typical blended raw water in Table 1 indicates that about 28 percent of the total hardness (TH) was due to magnesium (73 of 255 mg/l as $CaCO_3$). The target hardness of 100 mg $CaCO_3$/l, therefore, should have been achievable by leaving most of the magnesium in the water and by removing calcium hardness down to its minimum practical solubility of 30 to 40 mg $CaCO_3$/l. One concern, however, with this approach is the potential problem of magnesium silicate scaling in the high temperature service, such as boiler feedwater application, when the magnesium hardness exceeds 40 mg $CaCO_3$/l (6). Assuming magnesium hardness of approximately 70 mg $CaCO_3$/l is acceptable, the desired hardness should have been acheived by maintaining the softener pH at approximately 9.5 and by using conventional lime-soda ash softening operational parameters. Soda ash addition to remove non-carbonate hardness (NCH) could then be used to control the effluent total hardness.

Figure 2 is a probability plot of the softener effluent total hardness for all data collected during this phase of the study. As indicated by this figure, the target hardness of 100 mg $CaCO_3$/l was achieved only approximately 30 percent of the time. Effluent hardnesses of 125 mg $CaCO_3$/l or less were obtained approximately 70 percent of the time and 85 percent of the time the effluent hardness was 150 mg $CaCO_3$/l or less. It is believed the primary reason for the poor performance of the softening operation was the inability to adequately build up and retain solids in the reaction zone of the ClariFlow unit. Successful solids-contact softener operation is highly dependent upon having enough preformed crystals in the reaction zone to increase the rate of chemical reactions in the process. This point will be discussed further in a later section. Accurate control of pH in the range of 7.0 to 9.5 was impossible during the study due to problems with the pH controllers. The problem of maintaining pH control in this range made proper control of the softening process difficult and this lack of control may be part of the reason for poor overall performance. Because of the inability of the recarbonation unit to reduce the pH to desired levels during some operating conditions it was not possible to fully evaluate the effect of recarbonation on effluent hardness. As was discussed earlier, it was not possible to operate the filters during this portion of the study so the effect of filtration of the softener effluent was not obtained.

Figure 3 is a probability plot of the softener effluent total iron data collected during this phase of the study. This plot shows that the drinking water standard of 0.5 mg/l was achieved only 70 percent of the time. All dissolved iron values, however, were less than 0.2 mg/l and a total of 61 out of 72 data points for dissolved iron showed less than 0.1 mg/l of iron so effective filter operation should yield an effluent that would meet the standards without any difficulty.

Softener effluent total and dissolved manganese probability plots are shown in Figure 4. As can be seen from these plots, the drinking water standard limit of 0.05 mg/l was achieved 65 or 80 percent of the time depending on whether total or dissolved values were used. Manganese oxidation, which is normally the rate limiting step in the removal of manganese from potable water, proceeds rather slowly at pH's between 8.5 to 9.5. During those periods of the study when the neutralization stage was being operated at pH's less than 8.5 and because of the poor pH control achieved, effluent manganese values were often too high to meet the standards. It is entirely possible that filter effluent manganese values would be significantly lower than those observed due to the catalyzing effect of "seasoned" filters on manganese removal. As stated previously, however, it was not possible to evaluate this effect during the study.

Data were not collected routinely for those substances listed in the national interim primary drinking water regulations developed by EPA (7), but the results of one set of analyses (reported in Table 3) indicate that effluent from the plant could be expected to meet the standards for those substances that were measured. Not all of the substances included in the NIPDW regulations were included in the analyses performed.

Figures 5 and 6 are probability plots of the total dissolved solids and sulfates for all data collected during the entire study. These figures show the softener effluent met USPHS drinking water standards and the proposed EPA "Secondary Drinking Water Regulations for these constituents most of the time. No appreciable removal of sulfates were anticipated with the processes used. It should be pointed out again, however, that the raw water was not taken solely from Kittanning Run, the acid mine drainage impregnated stream, and hence, the raw water was lower in total dissolved solids and sulfates than had been anticipated.

Sludge Production Considerations. Because the major emphasis in this study was directed towards effluent quality, chemical usage, operational considerations, and cost, only general data on sludge production were collected. The process flow diagram of the plant shown in Figure 1 schematically illustrates the sludge handling procedures utilized. Underflow from both the neutralization stage sedimentation units and the ClariFlow softener units was thickened in a gravity thickener. Thickened sludge was then vacuum filtered and the resulting sludge cake was landfilled.

It was decided the best estimate of the amount of sludge produced and handled from the neutralization and softening stages could be obtained from a tally of the number of truck loads of filtered sludge hauled out for disposal. It was estimated from a limited amount of data on filter cake characteristics that each truck load contained approximately 2600 kg (5700 lbs) of dewatered cake (approximately 25 percent total solids). A total of 390 truck loads of sludge were hauled out over the period of the study from August 12 to December 8, 1974. The estimated sludge production from the overall process, therefore, was 690 kg (1520 lbs) of dewatered cake per 1000 cu m (.264 MG) of water processed. It was not possible to determine what fraction of the total sludge came from the neutralization stage and how much was generated in the softening operation. From the limited data available, vacuum filter yields ranged from approximately 37 kg (82 lbs) to 57 kg (126 lbs) of dewatered cake per square meter (10.8 ft^2) per hour. The sludge filtered well and no unusual operating problems were encountered.

Chemical Feed Considerations. The primary function of the neutralization stage during this study was to remove iron and manganese prior to the softening step. Iron removal did not present any particular problems; in fact, some carryover of iron floc from the first stage to the softener may have improved clarification in the softening step. On the other hand, manganese oxidation is highly dependent upon pH and the USPHS limits on manganese were not achieved during portions of the study. Four pre-softening stage conditions were investigated to determine the manganese removal capabilities of the system. Summary results for the different control conditions were as follows:

Control Condition	Manganese, mg/l	
	Total	Dissolved
Neutralization to pH 7.0	3.90	2.80
Neutralization to pH 9.0	.50	.08
Neutralization to pH 11.1	.20	.01
Neut. to pH 7.0 (10 mg/l $KMnO_4$)	.86	.10

The addition of potassium permanganate to the raw water influent line ahead of the neutralization stage is recommended in order to consistently assure oxidation of the manganese and its subsequent removal.

Magnesium removal in the first stage also was highly dependent upon pH as expected. Very little reduction in raw water magnesium (average value approximately 18 mg/l) was noted when the first stage pH was set at 7.0. Operation of the first stage at pH 9.0 to 9.5 resulted in a magnesium reduction of approximately 50 percent whereas operation at a pH 11.0 to 11.5 resulted in removal of nearly all of the magnesium.

Lime addition to the neutralization process did not contribute an equivalent amount of calcium hardness in the sedimentation basin effluent. Some calcium (approximately 15 mg/l) was apparently precipitated as calcium sulfate, as further evidenced by a comparable sulfate loss in the neutralization stage. This effect is of interest since it would tend to reduce effluent sulfate values and this would be particularly desirable for potable water use.

Softening Stage. Successful operation of a solids-contact softening unit of the type employed in this study requires the recycle of settled solids back to the reaction zone for intimate mixing of previously formed floc with the applied flow and added chemicals. The manufacturer's instructions called for controlling the slurry concentration at 5 to 10 percent by weight in the bottom of the basin in order to achieve optimal process operation (9). Except for a period of about three weeks during the study, however, it was not possible to maintain the slurry at the desired concentration. The general appearance of the softener effluent, for the most part, could be characterized as a turbid water containing a finely divided precipitate. Generally, samples drawn from various levels and zones within the softening unit failed to show any differential in the solids concentration which would indicate the desired solids buildup was occurring. It was possible to build the reaction zone solids up to the desired levels for brief periods when the pH in the softener was approximately 11.3 and the temperature of the water being treated was 12°C or higher. It was not possible to obtain good clarification when water temperatures were below 12°C. Likewise, good clarification was not obtained at pH 9.6 even when the water temperature was approximately 17°C. The reason for the good clarification only at the extremely high pH (11.3±) is not understood but it may be due to the virtually complete precipitation of magnesium as the hydroxide at that pH.

Even when good clarification was being achieved, the municipal water supply hardness objective of 100 mg $CaCO_3$/1 was not realized with the desired consistency. Several possible reasons for this higher-than-expected effluent hardness exist. Temperature-time relationships are significant in lime-soda ash softening. The theoretical reaction zone retention time for the softening unit used in the study was 25 minutes although actual retention times may have been less than that. No definitive retention time studies were made during the project, however, which would permit specific conclusions on this point. Assuming the actual retention time in the reaction zone was approximately 25 minutes, water temperatures should be at least 10°C in order to achieve the desired effluent hardness of 100. Table 4 shows the influence of temperature and time on effluent hardness for given chemical dosages.

Water temperatures of less than 10°C would be experienced in the plant studied commonly during winter months when Kittanning Run is being used as a source and hence, slow reaction times in the softener would make it impossible to achieve the desired effluent hardness of 100 mg $CaCO_3$/1 at the design flow of 13,200 cu m/day (3.5 MGD) per unit. Impoundment of this water source would tend to moderate and stablilize the temperature somewhat and should result in improved performance of all the treatment units.

Sulfate ion has been shown to have a major effect in slowing the rate of reaction and formation of calcium carbonate when less than 1000 mg/1 (0.1%) of preformed calcium carbonate is present (10). The water being treated in this study contained relatively high sulfates (see Fig. 7) and no bicarbonates. Hence, all the initial hardness to be removed was non-carbonate hardness (NCH). As indicated above, it was virtually impossible to maintain the desired level of reaction zone solids in the softener unit, even with internal sludge recycle, so that the condition necessary to effect optimum softening with this high sulfate water simply did not exist.

Furthermore, according to Symons (11), formation fo complex magnesium salts may be a problem with waters containing high NCH. These complex salts are more soluble than magnesium hydroxide; hence, theoretical hardness reduction may not be obtainable. The soda ash typically used to remove NCH in the lime-soda ash process tends to cause formation of basic carbonate complex salts such as Mg_4 $(OH_2(CO_3)_2$.

Under some conditions (temperature, presence of anions, complex salt formation, etc.) the precipitated salts formed in the softener may remain in colloidal form. This observation may account partially for the observed difficulty in obtaining good clarification during most of the study. In this study however, use of alum and polymers did not noticeable improve softener operation.

Chemical Utilization. An analysis of chemical feed data indicated that the neutralization of the influent water together with the desired pH adjustment for softening only required approximately 80 percent of the theoretical lime requirement. On the other hand, the soda ash requirement was approximately 15 percent greater than the theoretical demand. This observation is particularly significant as it shows that even with a 15-percent excess of soda ash, it was not possible to achieve the desired effluent hardness of 100 mg $CaCO_3$/l. Possibly, if this excess were increased, some additional hardness removal could be achieved but it could increase the chemical costs significantly.

Figure 8 shows the relationship between softener effluent hardness and the soda ash dosage for the data collected during the study. Although there is some scatter in the plotted points, the trend lines shown indicate the minimum attainable hardness under the conditions of the study was approximately 120 mg $CaCO_3$/l regardless of the soda ash dosage. As indicated earlier, waters containing appreciable amounts of NCH are difficult to soften and the data from this study support this observation.

The data and curves previously presented reflect operating conditions and results for the particular blend of waters from Impounding Dam and Kittanning Run used in the study. Different raw water sources or an appreciable change in the ratio of I.D. or K.R. would likely change the results obtained. Any such waters would probably be difficult to soften to low hardness.

Filter Operation. It is unfortunate that it was only possible to obtain filter operating data for approximately one month during the study. Data collected during this period generally indicated satisfactory performance with filter runs of 20 to 54 hours being observed. Filtration had little observable effect on hardness reduction but in several instances, it did significantly improve manganese removal. It was not possible, with the limited duration of the filtration study, to determine the long term effects of the poor clarifier performance on possible calcium carbonate buildup on the filter media, etc.; nor was it possible to fully evaluate the effect of filter "seasoning" on the removal of iron and manganese.

One major operating problem developed in the period October 16 to 22 when the filters first were placed in operation during a time when a polymer (Calgon 253) was being used in an attempt to improve softener clarification. The filters "fouled" very rapidly with a gelatinous material that coated the media. The filters were taken out of service and a variety of measures were taken to restore the media when normal backwash procedures failed to clean the media. The filters were again placed in operation after cleaning and operated for approximately one month with no evidence of fouling. It is believed the filter clogging, therefore, was directly a result of the polymer addition.

Economic Evaluation

An economic analysis of the cost of producing municipal water supply quality water for the period studied indicated the capital investment to be 3.35¢/cu.m. (12.70¢/1000 gal) and the operating and maintenance costs to be 6.43¢/cu.m. (24.37¢/1000 gal) in 1975 dollars.

Conclusions

A basic requirement of the study method utilized was that a relatively constant hydraulic 13,200 cu m/day (3.5 MGD) and mineral loading should be applied to the treatment process. In this regard, certain qualifying factors should be mentioned prior to discussing the major findings of the study. First, since the Kittanning Run (K.R.) source is an open channel flow, weather (rainfall) caused some variation in raw water quality. This condition could be controlled using K.R. exclusively or by blending the total flow in K.R. with constant quality Impounding Dam (I.D.) water, a fairly successful procedure so long as K.R. flow did not exceed the maximum 13,200 cu m/day (3.5 MGD). Temperature, however, was a parameter which could not be controlled.

Another equally important factor was the inability to obtain consistent softener operation with reaction zone solids and good clarification. This situation was a two-edged problem inasmuch as it was necessary to evaluate nontypical process performance of the unit as well as delay filter startup because of this condition. Data collection of this process was thereby limited for fear of prematurely fouling the filter media. The following conclusions were drawn from the study:

1. Coal mine drainage (average characteristics of pH 3.0, acidity 160 mg/l, iron 14.5 mg/l, and manganese 4.5 mg/l) was treated by the neutralization and lime-soda ash process to produce a finished water quality that will generally meet the EPA National Interim Primary Drinking Water Regulations.

2. The cost of producing a water suitable for blending with a potable supply from the subject drainage source is about 10¢/cu m (37¢/1000 gallons). This cost includes power, chemicals, operating personnel and maintenance.

3. Lime addition to the neutralization process did not contribute an equivalent amount of calcium hardness in the sedimentation basin effluent. Some calcium (approximately 15 mg/l) was apparently precipitated as calcium sulfate, as further evidenced by an equivalent sulfate loss, in the neutralization stage.

4. The softener unit operation employed was of the solids contact type that requires the return of settled solids to the reaction zone for intimate mixing of previously formed floc with the applied flow and chemical addition. The buildup of reaction zone solids (10-15% settleable solids by volume in 15 minutes) was a prerequisite for efficient reaction and effective clarification according to the manufacturer. With the exception of approximately three weeks during the study, it was impossible to obtain the recommended type of softener operation. The reason for failure of the unit to buildup solids it not understood.

5. The operating conditions under which reaction zone solids could be generated required a pH of 11.0 or higher. During these test periods the neutralization stage pH was 7.0 to 8.0. Heavy floc carryover from the sedimentation basin was characteristic of this first-stage operation. It was determined, however, that good softener operation at pH 11.3 also could be obtained without the presence of heavy floc carryover from the first stage. The favorable softener operation with a desired level of reaction zone solids was accomplished during periods when the water temperature was above 12°C. At water temperatures below 12°C, the desired softener process operation could not be reproduced.

6. The general appearance of the softener effluent for the most part could be characterized as a turbid, homogeneous quality of water containing a finely divided precipitate. Samples drawn from various levels and zones within the tank failed to show any distinct differentiation in the liquid contents which would indicate significant solids buildup.

7. The use of alum and coagulant aids (CA 233 and CA 253) was not effective in improving the coagulation - flocculation process in the softener.

8. The capacity of the recarbonation unit was not sufficient to lower the pH of the softener effluent below 9.5 when the softener process was operated at a pH in excess of 11.0. The carbon dioxide feed capacity was 682 kg/day (1500 lb/day) or about 51 g/cu m (428 lb/MG) of water to be treated.

9. A severe filter plugging problem was experienced during the period from October 23 to 28. It was suspected that the cause for this condition was due, at least in part, to coagulant aid fouling of the top 10 to 15 cm (4 to 6 in) of the filter media.

10. Settled solids in the thickener could not be effectively moved to the center collection well. The fluid characteristics of the sludge resulted in a condition whereby the solids accumulated in a ring approximately one third to half way from the periphery of the tank. Air lancing was partially successful in moving the sludge to the tank-center. Ultimately, it was necessary to lower the level in the thickener and hose the solids toward the middle sump.

Recommendations

It should be clearly understood that the treatment units and the process discussed in this report were not designed or intended to treat the Kittanning Run source of mine drainage to a degree better than that required for stream release, i.e., pH 6.0 - 9.0 and iron 7.0 mg/l or less. However, having the opportunity to observe the complete system with Kittanning Run as the raw water source has provided valuable operating experience relative to applying better treatment if the need arises. It is in reference to this possible application, as well as other general problem areas, that the following recommendations are made:

1. Conventional design criteria will generally apply to the softening process (lime/soda ash) so long as the water temperature remains above 12°C (54°F). It is assumed that if Kittanning Run or a similar quality of raw water were to be treated, impoundment would be provided, so in all probability, temperature below this

threshold level would not be experienced.

Turbulence should be avoided at transfer points in the system after mixing and particularly ahead of the clarifier to minimize floc breakup.

3. The use of potassium permanganate should be limited to head-end of the plant application. This study indicated that the addition point ahead of neutralization (7 hours before the softening unit) was best. Potassium permanganate feed directly to the softener was very difficult, if not impossible, to control.

4. Application of the particular solids contact type of softener unit used in this study appears questionable.

5. Design of the thickener unit operation for handling a mixture of iron hydroxide/calcium carbonate solids should take into consideration the tendency of this type of sludge to "doughnut," or resist movement to the tank center. The bottom slope in the unit available for the study was about 14.5 percent. Increasing the floor slope would facilitate sludge removal and solids control.

6. Provision should be made for feeding coagulant aid to the sedimentation basin (ahead of softening) inlet line. This capability is important because of the characteristic light iron hydroxide floc formed in the neutralization stage. In general, flexibility in chemical addition points is most desirable.

7. The data analysis indicated that although on occasion a softener effluent hardness of less than 100 mg/l $CaCO_3$ was obtained, an average of 120 mg/l could be expected, regardless of the amount of soda ash fed. This result must be interpreted with some caution because the process condition did not afford control of temperature, softener reaction zone solids, or continuous filter operation. Where a specific application of this treatment process is being considered, it would be advisable to determine this minimum achievable hardness level in the laboratory.

Acknowledgement

The physical plant which was the subject of this study was constructed with "Operation Scarlift" funds, the $500 million conservation bond issue program administered by the Commonwealth of Pennsylvania through the Department of Environmental Resources. The actual study was performed for DER under grant agreement with the U. S. Environmental Protection Agency. Support of the project by these agencies is gratefully acknowledged.

Data collection, evaluations and the preparation of this report were under the direct supervision of technical staff of Gwin, Dobson & Foreman, Inc. and Dr. David A. Long, Associate Professor of Civil Engineering at the Pennsylvania State University.

The splendid cooperation provided by the City of Altoona Treatment Plant Employees and their supervisor, Mr. David Barr in collecting samples, exercising the operational control, and, in general, providing the overall work effort essential to conducting a project of this type

is gratefully acknowledged.

References

1. The National Safe Drinking Water Act, Public Law 93-523 (December 16, 1974).

2. Public Health Service, Drinking Water Standards, 1962, U. S. Dept. of Health, Education, and Welfare, Public Health Service, Washington, D. C. (1962).

3. Methods for Chemical Analysis of Water and Wastes, U. S. Environmental Protection Agency, Technology Transfer, Washington, D. C. (1974).

4. Standard Methods for the Examination of Water and Wastewater, American Public Health Association, 13th Edition (1971).

5. Handbook for Monitoring Industrial Wastewater, U. S. Environmental Protection Agency, Technology Transfer, Washington, D. C. (1973).

6. Larson, R. E., Lane, R. W., and Neff, C. H., Stabilization of Magnesium Hydroxide in the Solids Contact Process, Journ. AWWA 51:12: 1551 (December, 1959).

7. National Interim Primary Drinking Water Regulations, Federal Register, 40:248:59566 (December 24, 1975).

8. Penn Environmental Consultants, Inc., Pittsburgh, PA, unpublished data (1974).

9. General Operating Instructions, Walker HC ClariFlow Series, No. 6-330-1172, P.L. 6060, Walker Process Equipment.

10. Calise, V. J., Duff, J. and Dvorin, R., Chemical Reactions in Hot and Cold Treatment Units, Journ. AWWA, 47:7:665 (July, 1955).

11. Symons, G. E., Water Softening - Part 2 - The Lime-Soda Process, Water & Sewage Works, 104:11:496 (November 1957).

TABLE 1

Typical Blended Raw Water Characteristics
Kittanning Run : Impounding Dam = 1 : 1.33

Parameter	Value
pH	3.0
Acidity, mg/l as $CaCO_3$	170
Calcium, mg/l	28
Magnesium, mg/l	18
Iron, mg/l	17
Manganese, mg/l	4.5
Aluminum, mg/l	13
Sodium, mg/l	1.8
Hardness, mg/l as $CaCO_3$	260
Sulfates, mg/l	270
Specific conductance, umhos/cm^2	820

TABLE 2

Process Control Conditions for Municipal Water Supply Quality Phase

Period	Target pH		
	Neut. Stage	Soft Stage	
Aug. 12-19, 1974	11.0	11.0	Recarbonation; no filtration; tried Calgon 233; no measurable reaction Zone solids.
Sept. 1-15	9.5	11.0	Recarbonation; no filtration; alum addition to softener; no measurable reaction zone solids.
Sept. 16-19	7.0	10.5	Recarbonation; no filtration; no measurable reaction zone solids.
Sept. 20-29	7.0	11.4	Recarbonation not effective; no filtration; reaction zone solids Started to buildup.
Oct. 1-4	7.0	11.4	No recarbonation or filtration, modified operation of aeration tank To investigate effect of floc carry-over. Reaction zone solids 14% By volume.
Oct. 5-7	7.0	11.5	Same operation as Oct. 1-4 except added Calgon 233; reaction zone Solids 13%.
Oct. 8-10	7.0	11.3	No recarbonation or filtration, reduced aerator operation; reaction Zone solids 12%.
Oct. 12-16	9.5	11.3	Same operation as Oct. 8-10. No measurable reaction zone solids. Tried Calgon 253 on 11/16.
Nov. 25-30	8.5	10.5	Recarbonation (2 days); filtration
Dec. 3-8	7.0	11.3	Recarbonation (4 days); filtration (2 days)
June 16-26, 1975	9.0	9.6	Recarbonation; no filtration. No measurable reaction zone solids.

TABLE 3

Water Quality Data Comparison with EPA National Interim Primary Drinking Water Regulations

Parameter(1)	Raw Water Kitt. Run	 Imp. Dam	Neutralization stage	Recarb. basin eff.	EPA NIPDWR (7)
Arsenic	0.030	0.030	0.030	0.030	0.05
Barium	0.05	0.05	0.05	0.05	1.
Cadmium	0.004	0.004	0.004	0.004	0.010
Chromium	0.010	0.010	0.010	0.010	0.05
Cyanide	0.005	0.005	0.005	0.005	0.2
Lead	0.030	0.030	0.030	0.030	0.05
Mercury	--	--	--	--	0.002
Nitrate	--	--	--	--	10.0
Selenium	0.010	0.010	0.010	0.010	0.01
Silver	--	--	--	--	0.05
Fluoride	0.33	0.19	0.29	0.29	1.4 - 2.4
Copper	0.3	0.020	0.010	0.010	1.0*
Zinc	1.1	0.15	0.070	0.030	5.0*

NOTE: All values are mg/l; the test results are from Reference 8.

*1962 USPHS Drinking Water Standards and proposed "Secondary Drinking Water Standards."

TABLE 4

Residual Hardness Observed at Different Temperatures (mg/l as $CaCO_3$(

Time (min.)	Temperature - °C 2	 10	 20
30	124	84	64
60	82	71	46
90	71	67	46
120	75	62	--

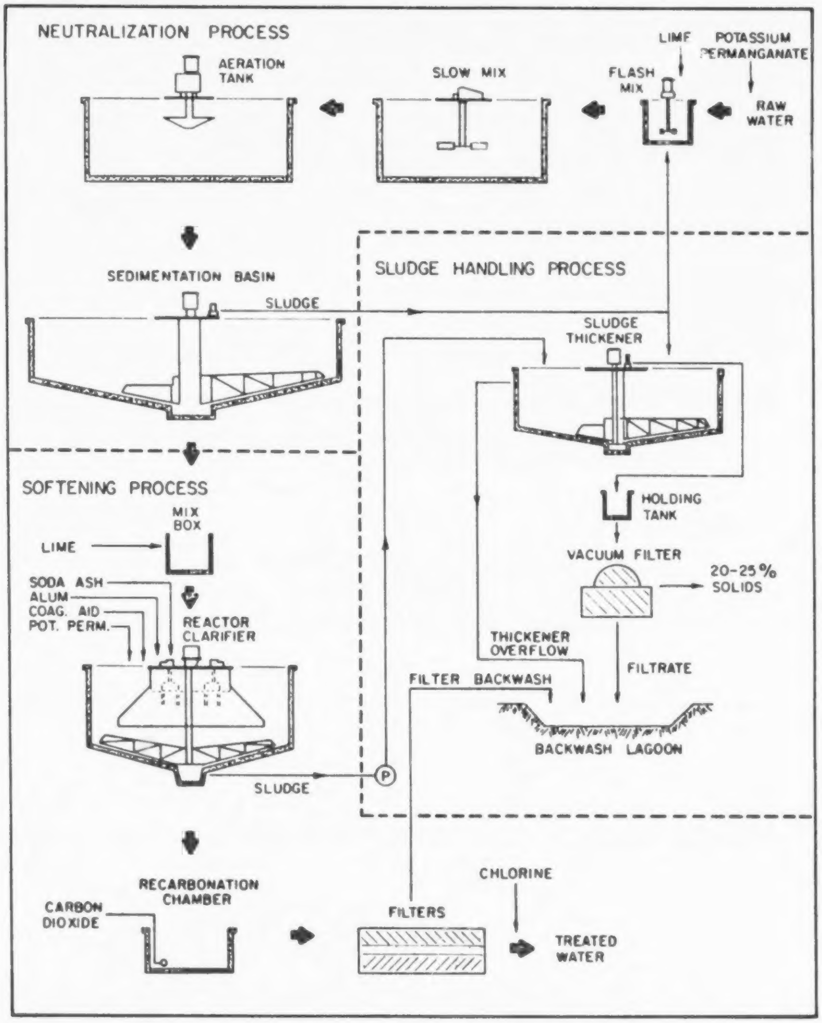


FIGURE I. DIAGRAM OF THE ALTOONA TREATMENT SYSTEM ILLUSTRATING UNIT PROCESSES

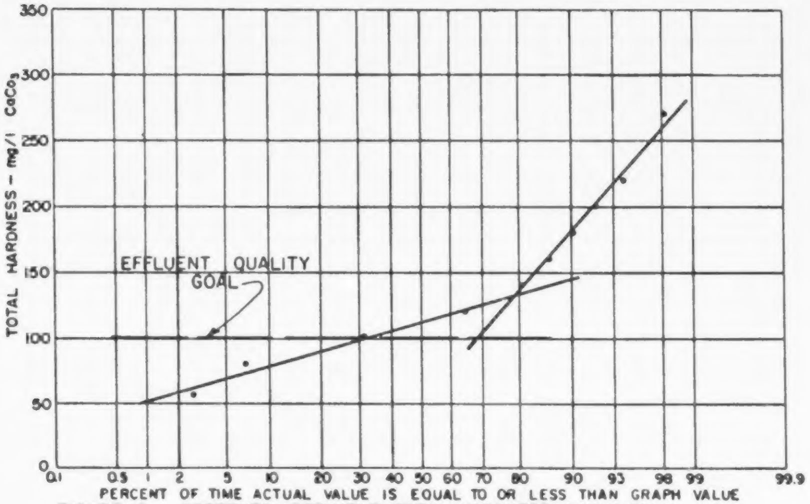


FIGURE 2 : SOFTENER EFFLUENT TOTAL HARDNESS MUNICIPAL WATER SUPPLY QUALITY PROBABILITY PLOT

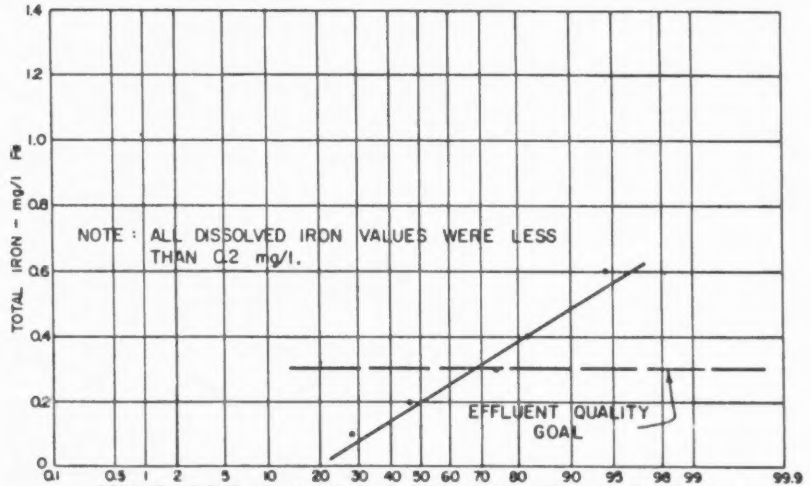


FIGURE 3 : SOFTENER EFFLUENT TOTAL IRON MUNICIPAL WATER SUPPLY QUALITY PROBABILITY PLOT

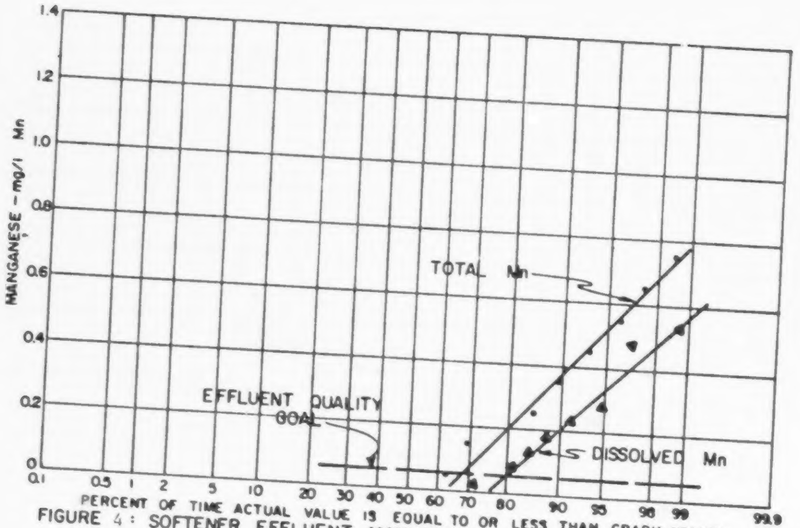


FIGURE 4 : SOFTENER EFFLUENT MANGANESE MUNICIPAL WATER SUPPLY QUALITY PROBABILITY PLOT

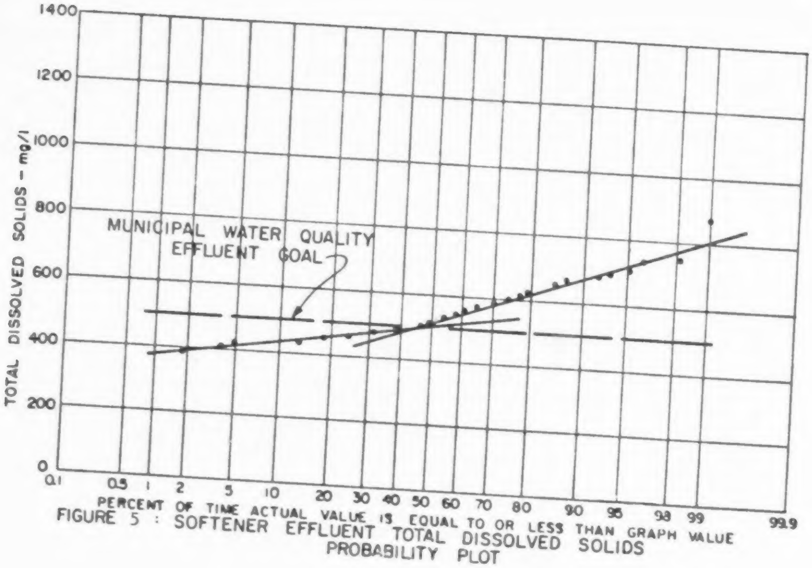


FIGURE 5 : SOFTENER EFFLUENT TOTAL DISSOLVED SOLIDS PROBABILITY PLOT

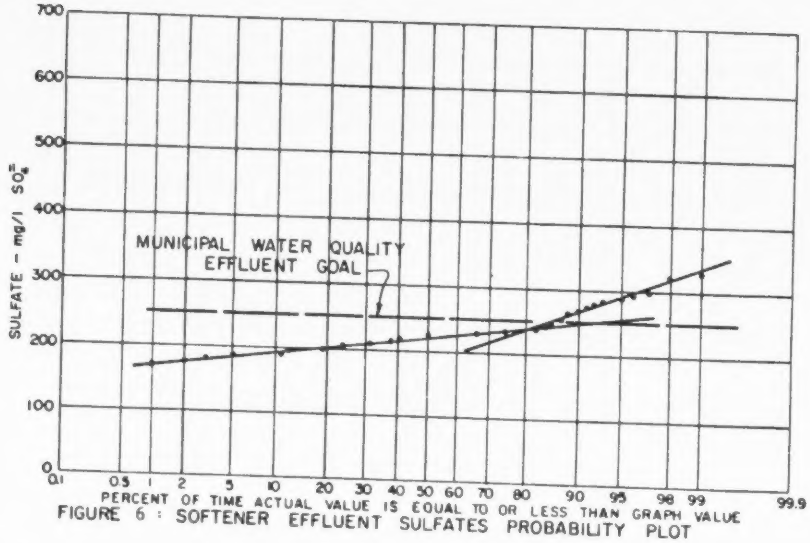


FIGURE 6 : SOFTENER EFFLUENT SULFATES PROBABILITY PLOT

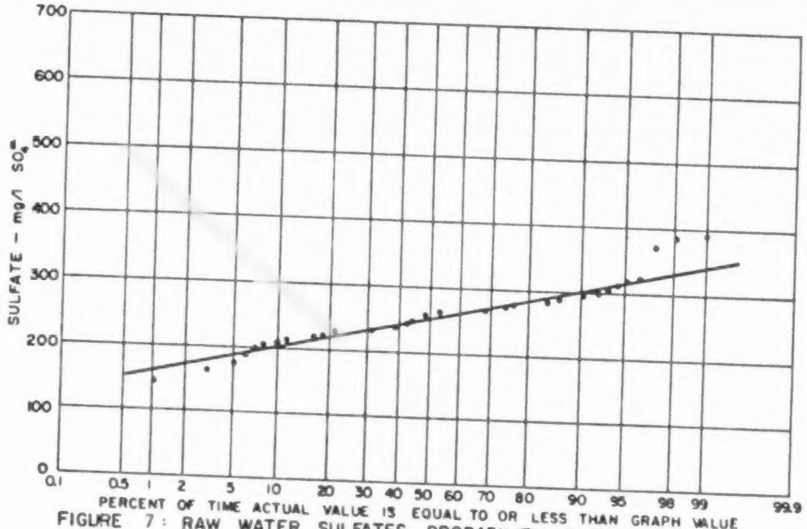


FIGURE 7 : RAW WATER SULFATES PROBABILITY PLOT

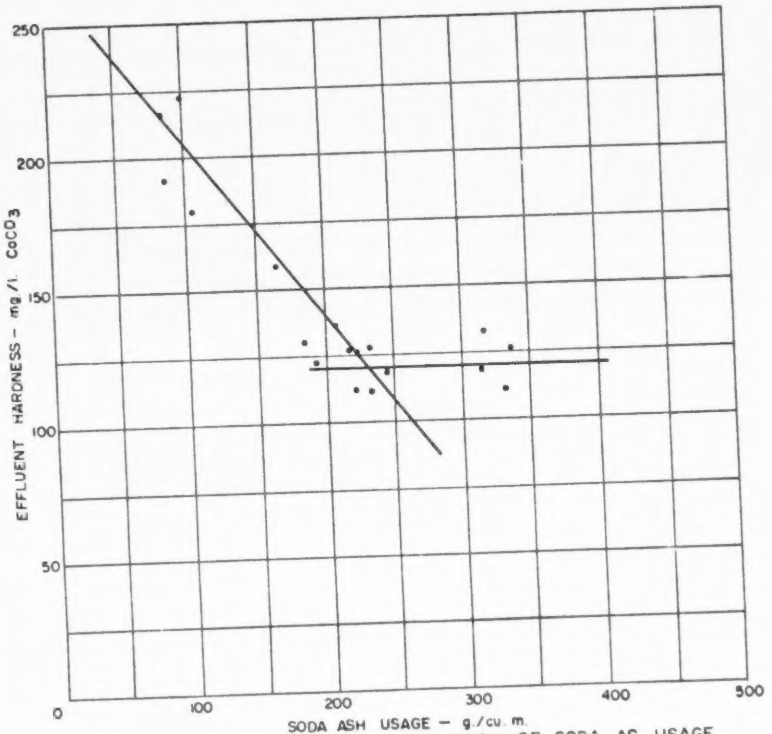


FIGURE 8 : EFFLUENT HARDNESS AS A FUNCTION OF SODA AS USAGE

NITROGEN REMOVAL IN MODIFIED SUBSURFACE SEWAGE DISPOSAL SYSTEM

Aldo Andreoli
Principal Public Health Engineer
Bureau of Environmental Health Services
Suffolk County Department of Health Services
Hauppauge, New York

Nicholas Bartilucci
Principal Public Health Engineer
William F. Cosulich Associates, P.C.
Woodbury, New York

Robert Forgione
Project Engineer
William F. Cosulich Associates, P.C.
Woodbury, New York

Royal Reynolds
Assistant Public Health Engineer
Bureau of Environmental Health Services
Suffolk County Department of Health Services
Riverhead, New York

INTRODUCTION

Suffolk and Nassau Counties on Long Island, New York, depend on groundwater for their entire water supply. Long Island is situated on the Atlantic Ocean at the southeastern end of New York State. Suffolk County occupies about 2,390 square kilometres (922 square miles) on eastern Long Island, and Nassau County occupies about 750 square kilometres (291 square miles) to the west of Suffolk. Queens and Kings (Brooklyn) Counties, occupying the remaining 500 square kilometres (193 square miles) of Long Island, are part of New York City; although partly dependent on the groundwater, these two counties receive most of their water from New York City's upstate surface water system.

Suffolk County's population is about 1,280,000. Less than 200,000 persons, or about 16% of the population, is sewered, the remainder being dependent on individual subsurface sewage disposal systems. All new residences in unsewered areas of Suffolk County are required to have a conventional sewage disposal system consisting of a septic tank and leaching pool(s). Each new system is inspected and must be approved by the Department of Health Services prior to issuance of a Certificate of Occupancy. By the end of 1976 there were approximately 300,000 individual home subsurface sewage disposal systems in Suffolk County; over 6,000 new systems were approved in 1976 alone. By 1995 Suffolk is expected to have a population of about 1,750,000 and it is expected that the vast majority of these people will still be dependent on individual disposal systems.

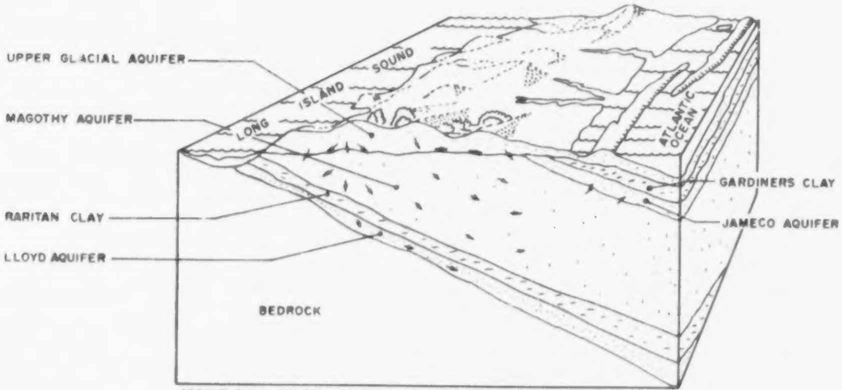
Nassau County's population is about 1,460,000. About 910,000 persons, or 62% of the population is sewered, the remainder being dependent on individual disposal systems. By 1995 Nassau County is expected to have a population of about 1,490,000; although most of these people will be sewered, the remainder will still be dependent on individual disposal systems.

Long Island Geology

The soils below the surface of Long Island are composed of gravel, sand, silt, and clay deposits. The deposits form the aquifers and confining strata of Long Island's groundwater system. Aquifers, from which water can be readily drawn, are composed predominantly of sand and gravel. Clay and silt deposits separate the aquifers but do not completely prevent vertical and horizontal movement of water between them.

Major aquifers and confining layers are shown in a generalized cross-section in Figure 1. The aquifers are, from the land surface downward, the upper glacial aquifer, the Magothy aquifer and the Lloyd aquifer. The confining layers are the Gardiners Clay and the Raritan Clay.

Also note the general direction of groundwater movement depicted in Figure 1. Contaminants entering the aquifer in the central area may affect the entire groundwater system, which contaminants entering along the shoreline may pollute shallow groundwater and may be carried to the bays or sound along with the groundwater flow.



ARROWS SHOW GENERAL DIRECTION OF GROUNDWATER MOVEMENT

MAJOR HYDROGEOLOGICAL UNITS AND

GROUNDWATER MOVEMENT ON LONG ISLAND, NEW YORK[1]

FIGURE Nº 1

WATER BUDGET

The U.S. Geological Survey has defined a "water budget area" of Long Island. This water budget area comprises almost 2000 km^2 (760 sq miles) of Nassau and Suffolk Counties.[1] Boundaries chosen for this area are ones within which estimates of groundwater inflow and outflow can be determined with some degree of confidence to aid planners concerned with areawide management of the groundwater resource.

An average of 112 cm/y (44 in/yr) of precipitation, or 6.1×10^6 m^3/d (1,600 mgd) fall on the water budget area. This precipitation is then distributed as follows:[1]

Evapotranspiration	2.9×10^6 m^3/d	(760 mgd)
Stream flow from runoff	0.1×10^6 m^3/d	(20 mgd)
Groundwater recharge	3.1×10^6 m^3/d	(820 mgd)
Total	6.1×10^6 m^3/d	(1,600 mgd)

Under natural conditions, a quantity of water equal to the amount of groundwater recharge would move out of storage and thus keep the groundwater system balanced. This movement of water would be as follows:[1]

Stream flow from groundwater	1.2×10^6 m^3/d	(320 mgd)
Subsurface outflow	1.8×10^6 m^3/d	(470 mgd)
Groundwater evapotranspiration	0.05×10^6 m^3/d	(15 mgd)
Spring flow	0.05×10^6 m^3/d	(15 mgd)
Total	3.1×10^6 m^3/d	(820 mgd)

However, these natural conditions no longer exist on Long Island. Man-made wells draw water from the aquifers for industrial, commercial, agricultural, and domestic purposes. Groundwater withdrawal in Nassau and Suffolk Counties totaled approximately 1.3×10^6 m^3/d (350 mgd) in 1975.[2] Some of this water is returned to the ground. That portion not returned is termed "consumptive use." Consumptive use in Nassau and

Suffolk Counties has been estimated to be approximately 0.7 x 10^6 m^3/d (180 mgd) for the period 1969 to 1973.[2] The major contributor to consumptive use is sewer outfalls, which convey "used" water to the bays, ocean, or sound.[2] Since 1973, additional areas of both Counties have been, or will soon be, serviced by sewer outfalls.

In areas not served by sewer outfalls, "used" water is returned to the ground. On the basis of data given by the New York State Department of Health,[3] it is estimated that more than 0.4 x 10^6 m^3/d (100 mgd) of domestic wastewater is returned to the ground via subsurface disposal systems (leaching systems). This practice helps maintain the natural condition of groundwater quantity. However, because leaching systems as presently designed are not capable of removing all pollutants from wastewater, this practice contributes to groundwater pollution.

GROUNDWATER POLLUTION

In general, Long Island groundwater meets federal and state drinking water standards. Where water treatment has been necessary, it usually has consisted of iron removal or sequestering, adjustment of naturally low pH, or chlorination, practiced by some suppliers as a precautionary measure against bacterial contamination within the distribution system.

Chemical contamination of the water supply has become an increasing problem on Long Island because refractory or biologically stable substances such as organics, nitrates, and synthetic detergents leach through the soil and reach the groundwater. These substances can cause aesthetic problems or, more seriously, can pose public health threats. Organics in significant quantities are not usually found in household wastewater. Detergents do not seem to be a problem in deep public water supply wells and, to prevent such pollution, sale of certain detergents has been banned in Suffolk County. Of particular concern for public water supply as it relates to wastewater from household disposal systems is nitrate contamination.

Nitrate is of public health concern because in sufficient concentration, it can cause methemoglobinemia in infants. Because of this, the U.S. Environmental Protection Agency (EPA) Primary Drinking Water Regulations and Part V of the New York State Sanitary Code set a limit on the amount of nitrate in drinking water. This limit is 10 mg/l as nitrogen.

A major contributor of nitrate to groundwater is effluent from conventional subsurface wastewater disposal systems. It has been estimated that such systems contribute almost 3 x 10^6 kg (3,400 tons) of the more than 15 x 10^6 kg (17,000 tons) of nitrogen leached to Long Island groundwater each year.[4] Other sources of nitrates in groundwater include fertilization of crops and lawns, sewer exfiltration, recharge of storm runoff, and recharge of wastewater treatment plant effluent.[3-5]

Data available through 1976 indicate that in Nassau County, where much of the population was unsewered for many years, 35% of 447 public water supply, private, and monitoring wells showed increasing nitrate trends while only 7% showed decreasing nitrate trends. In 1976, 19 of Nassau County's 386 public water supply wells had nitrate levels exceeding the public health limit of 10 mg/l of nitrate as nitrogen. In order to safeguard public health, such wells must either be shut down or their water must be treated or blended with higher quality water before distribution.[6] In Suffolk County several public water supply wells with high nitrate concentration have been deepened or their water has been blended with higher-quality water to obtain water with lower nitrate concentrations.

Individual home subsurface disposal systems are designed to dispose of liquid waste. Mechanisms involved in the removal of various contaminants have not been well documented nor are they well understood. Suffolk County Department of Health Services, recognizing the need for improving individual disposal systems to reduce the nitrogen in leachate reaching the groundwater, undertook a study to develop such a system.

SCOPE AND OBJECTIVES OF STUDY

The study encompassed the design, construction, and operation of a full-scale system consisting of a conventional septic tank-leaching field wastewater disposal system combined with a subsurface system using natural soil treatment mechanisms to accomplish nitrogen removal from disposal system leachate. Immediate study objectives were to further the understanding of the operation of a typical septic tank-leaching field disposal system, increase the knowledge of the subsurface nitrification-denitrification phenomena, and make recommendations for further system development.

The long-range objective is to develop a nitrate-nitrogen removal system that could be built during new home construction. For the system to be practical it must

- Nitrify septic tank effluent,
- Remove nitrate nitrogen before the effluent reaches groundwater,
- Be cost-effective compared to other acceptable waste treatment and disposal alternatives,

- Preferably be free of mechanical equipment so that frequent maintenance by the user is not necessary,

- Preferably not require electrical energy that may be interrupted,

- Comply with general surface area requirements currently used for individual wastewater disposal systems, and

- Have a life expectancy at least equivalent to that of current individual wastewater disposal systems.

THEORETICAL CONSIDERATIONS

Many papers have been written on the subject of the nitrogen cycle and the processes responsible for this cycle. Presented below is a summary of those facets of the nitrogen cycle that are relevant to this study. Additional information is available in the literature.[7-13]

During life processes, nitrogen compounds are released from animal bodies through urine and feces. Urine contains nitrogen resulting from the metabolic breakdown of protein. The nitrogen in urine exists principally as urea, which hydrolyzes rapidly to ammonium carbonate. Feces contain appreciable amounts of nitrogen as protein. This protein converts to ammonia (ammonium ion) by bacterial action under either aerobic or anaerobic conditions. It is therefore not surprising that nitrogen in septic tank effluent appears predominantly as ammonia.

For the purposes of this study, the key processes governing the transformation of nitrogen in the soil are nitrification and denitrification. Brief descriptions and simplified equations of these processes are given below.

Nitrification. Under aerobic conditions ammonia is transformed to nitrite by the Nitrosomonas group of bacteria, as given by the equation:

$$NH_4^+ + 1.5O_2 \xrightarrow{\text{bacteria}} NO_2^- + 2H^+ + H_2O \qquad (1)$$

Under continued aerobic conditions nitrite is rapidly transformed to nitrate by the Nitrobacter group of bacteria, as given by the equation:

$$NO_2^- + 0.5O_2 \xrightarrow{\text{bacteria}} NO_3^- \qquad (2)$$

The overall process by which ammonia is transformed to nitrate is termed "nitrification," and can be represented by combining Equations 1 and 2 above as follows:

$$NH_4^+ + 2O_2 \xrightarrow{bacteria} NO_3^- + 2H^+ + H_2O \qquad (3)$$

The hydrogen ion further reacts to destroy bicarbonate alkalinity, which is naturally present in the wastewater, according to the following equations:

$$H^+ + HCO_3^- \rightleftharpoons H_2CO_3 \qquad (4)$$

$$H_2CO_3 \rightleftharpoons CO_2\uparrow + H_2O \qquad (5)$$

Stoichiometrically, 7.14 mg of alkalinity as $CaCO_3$ are destroyed when 1 mg of ammonia-nitrogen is transformed to nitrate-nitrogen.

Denitrification. After nitrification, if the wastewater is subjected to conditions of low dissolved oxygen (DO), then the nitrate-nitrogen and residual nitrite-nitrogen can be reduced to nitrous oxide, N_2O, or free nitrogen, N_2. The reductions are brought about predominantly by facultative bacteria, and take place only if there is an adequate supply of biologically oxidizable carbon. In this study, methanol was used to decrease the DO concentration biologically and to provide a source of carbon. The following discussion relates to the use of methanol.

Reduction of nitrate to nitrogen gas occurs in two steps. In the first step nitrate is transformed to nitrite as follows:

$$NO_3^- + 0.33CH_3OH \xrightarrow{bacteria} NO_2^- + 0.33CO_2\uparrow + 0.67H_2O \qquad (6)$$

In the second step nitrite is transformed to nitrogen gas as follows:

$$NO_2^- + 0.5CH_3OH \xrightarrow{bacteria} 0.5N_2\uparrow + 0.5H_2O + OH^- + 0.5CO_2\uparrow \qquad (7)$$

The overall process by which nitrate-nitrogen is transformed to nitrogen gas is termed "denitrification," and can be represented by combining Equations 6 and 7 above:

$$NO_3^- + 0.83CH_3OH \xrightarrow{bacteria} 0.5N_2\uparrow + 0.83CO_2\uparrow + 1.17H_2O + OH^- \qquad (8)$$

This simplified equation does not include formation of nitrous or nitric oxides, nor does it include synthesis of bacteria.

Equation 8 shows that denitrification using methanol results in production of the hydroxyl ion, OH^-. As a result, stoichiometrically 3.57 mg of alkalinity as $CaCO_3$ are produced when 1 mg of nitrate-nitrogen is transformed to nitrogen gas.

DESIGN AND INSTALLATION

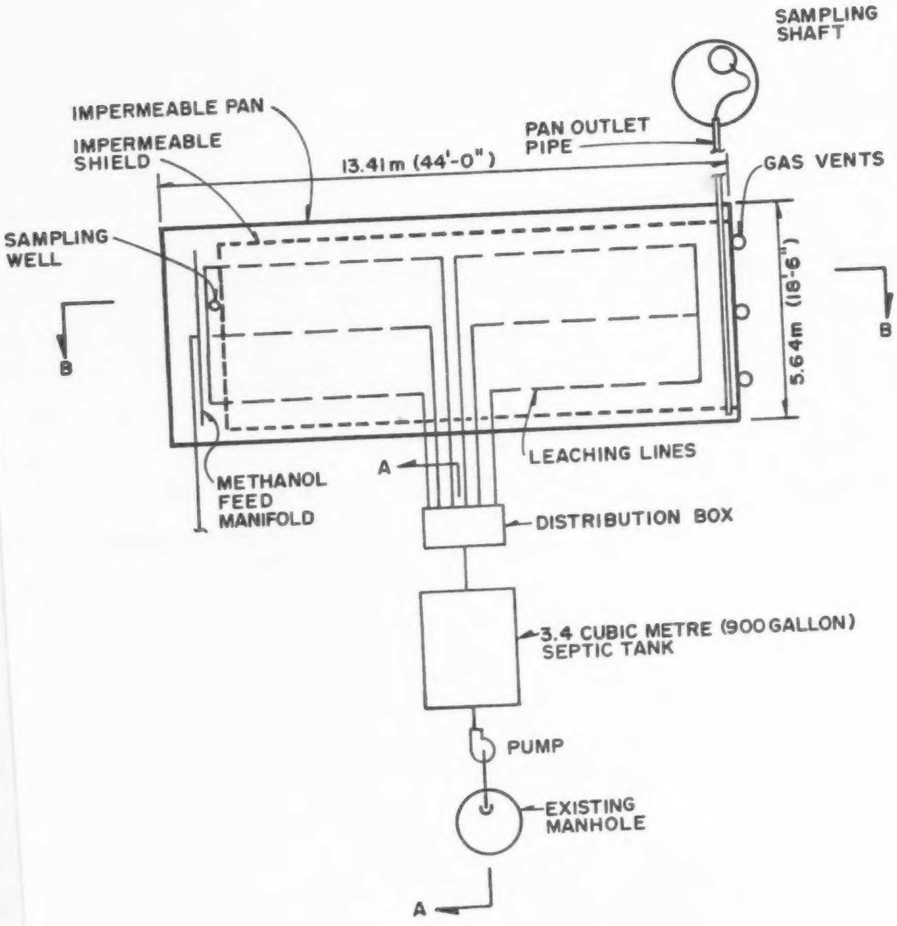
A full-scale experimental household disposal system, consisting of a septic tank, leaching field, impermeable shield, and pan-shaped impermeable membrane, was constructed on the grounds of Brookhaven National Laboratory in Suffolk County. Means of sampling flow were built into the system during construction and are described below. Figure 2 shows a plan of the system, and Figures 3 and 4 show sections through the system and locations of sampling points.

Disposal system. A 1.8-m x 1.8-m x 2.1-m-high (6-ft x 6-ft x 7-ft-high) prefabricated shed was placed on grade between the septic tank and a sewer manhole serving a nearby apartment complex. This shed housed a 0.0016 m^3/s (25 gal/min) wastewater pump, methanol feed equipment, electrical controls, and miscellaneous supplies.

The pump was controlled by automatic timers to provide pumping schedule flexibility. The 2.54-cm (1-in.) polyvinyl chloride (PVC) pump suction line extended underground from the shed into the sewer manhole. A 30.5-cm (12-in.) high dam was constructed in the manhole to provide a reservoir for the pump suction. A coarse screen was placed around the suction line to prevent solids larger than 1.6 cm (0.625 in.) (disposable diapers, for example) from clogging the pump. A check valve was installed in the suction line to keep the pump primed. The pump discharge line branched into two diaphragm valve-controlled discharge lines, one to feed the desired flow to the septic tank and the other to return excess flow to the sewer manhole.

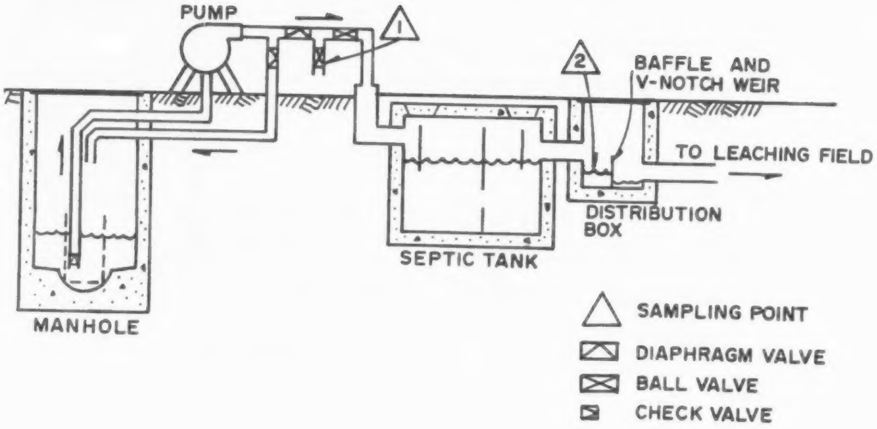
The standard 3.41-m^3 (900-gal) capacity septic tank was followed by a distribution box with a single inlet, a baffle, and six outlets leading to the leaching field. A 30-deg V-notch weir was installed in the distribution box baffle, and a level recorder was installed to record continuously the liquid level upstream of the weir.

The leaching field was of standard design and consisted of three 12.19-m-(40-ft-) long leaching lines, spaced 1.83 m (6 ft) apart, laying in a 4.88-m x 12.19-m x 33-cm (16-ft x 40-ft x 13-in.) bed of washed gravel.



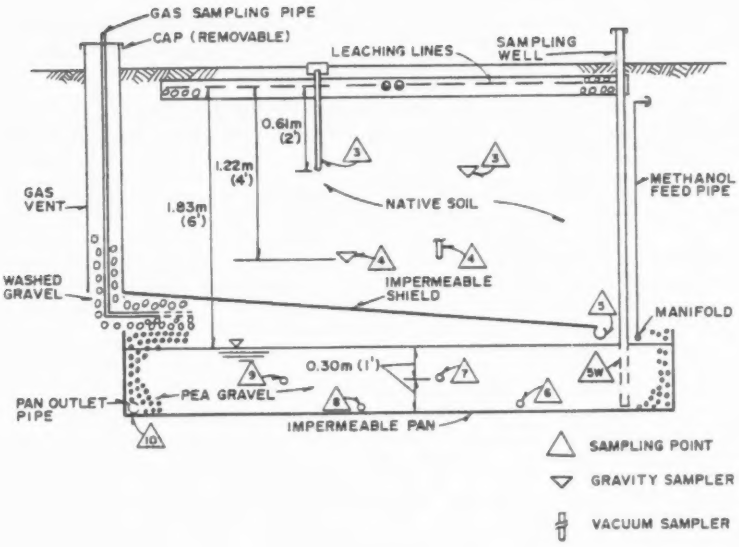
SITE PLAN SHOWING ALL FACILITIES

FIGURE 2



SECTION A-A THROUGH MANHOLE, PUMP, SEPTIC TANK AND DISTRIBUTION BOX

FIGURE 3



SECTION B-B THROUGH LEACHING FIELD AND PAN

FIGURE 4

An impermeable shield of corrugated fiberglass was installed about 1.52 m (5 ft) below the leaching field. This depth was chosen to provide a length of travel through the unsaturated soil in which essentially complete nitrification of leachate-ammonia could occur.[3, 14] The shield sloped from the discharge end of the pan down to the influent end and was underlain by a bed of washed gravel, as shown in Figure 4. The shield was intended to

- Prevent short-circuiting of leachate through the pan by directing it to one end and forcing it to travel through the entire length of the pan to the discharge end;
- Prevent the gaseous end products of denitrification from rising through and displacing oxygen from the unsaturated soil below the leaching field; and
- Provide a location at which methanol can be added conveniently to the leachate entering the pan.

Three 30.5-cm- (12-in.-) diam gravel-filled vent pipes were installed at the high end of the shield to vent the gases produced by the denitrification process. Caps were provided for these vent pipes, but were not installed during initial system operation.

The impermeable pan was installed below the shield to detain the leachate and provide a location where conditions favorable to denitrification could be developed. The pan was constructed of fiberglass lined with a 0.51-mm (20-mil) plastic liner, measured 5.64 m x 13.41 m x 0.76 m deep (18.5 ft x 44 ft x 2.5 ft deep), and was filled with pea gravel.[7] It was designed wider and longer than the leaching field gravel bed to ensure collection of leachate, which was expected to exhibit limited horizontal movement as it percolated downward.[3] The pan depth provided for a liquid depth of 0.61 m (2 ft), plus 0.15 m (6 in.) of freeboard. Given the pan length and width stated above, a 0.61-m (2 ft) gravel filled depth was intended to provide a 10-day liquid detention period at a wastewater flow rate of 1.514 m^3/d (400 gal/day).[7, 15, 16] A 7.6-cm- (3-in.-) diam perforated PVC pipe was installed along the bottom of the discharge end of the pan and extended through the pan into the sampling shaft described below, to serve as the pan outlet.

A sampling shaft 2.44 m (8ft) in diameter and 4.88 m (16 ft) deep was constructed next to the disposal system at the discharge end of the impermeable pan. This shaft contained collection tubes from sampling devices installed in the system as well as a vacuum system required for some of the samplers.

Flexible tubing connected the pan outlet pipe to a 0.21-m^3 (55-gal) drum in the shaft. The flexible tubing was arranged such that it controlled the liquid depth, and thus the liquid detention time, within the pan. A float-controlled submersible pump was installed to discharge a fixed quantity of treated wastewater from the drum each pumping cycle. An event counter was installed to keep track of the number of pumping cycles to enable calculation of pan discharge flow rate.

A methanol feed system was installed to feed an aqueous solution of methanol to the pan automatically. The system consisted of three 0.0038-m^3 (1-gal) containers, controls for automatic discharge of the contents of one container daily, and a manifold at the influent end of the pan. The manifold, shown in Figures 2 and 4, was PVC pipe with a series of holes along its bottom, sized and spaced to allow uniform distribution of the methanol solution across the width of the pan. A second manifold (not shown in the figures) with larger holes was installed 0.61 m (2 ft) above the first manifold to serve as backup in case the first manifold clogged. Flexibility was incorporated in the design of the system to allow feeding of other liquid substances into the pan.

Sampling points and sampling devices. Sampling points were provided throughout the disposal system to permit sample withdrawal for subsequent laboratory analysis. Representative samples from these points were necessary to assess the operation of this system. The sampling points and devices described below were chosen with this need in mind. The sampling points and their locations within the system are as follows:

Sampling Point	Location
1	Wastewater pump discharge
2	Distribution box
3	0.61 m (2 ft) below the leaching field
4	1.22 m (4 ft) below the leaching field
5	1.83 m (6 ft) below the leaching field
5W	Head of pan
6	0.166 point in pan, bottom of pan
7	0.333 point in pan, mid-depth in pan
8	Half point in pan, bottom of pan
9	0.666 point in pan, mid-depth in pan
10	Pan outlet

Sampling Point 1 consisted of a valved tap on the pump discharge pipe to the septic tank. Samples from this point were referred to as "raw wastewater" or "septic tank influent."

Sampling Point 2 was at the upstream side of the distribution box baffle where the dam formed by installing the V-notch weir in the baffle created a standing pool of septic tank effluent about 15 cm (6 in.) deep. Samples scooped from here were termed "septic tank effluent."

Sampling Point 3 consisted of six sampling devices installed in unsaturated soil 0.61 m (2 ft) below the leaching line closest to the distribution box. Two of these devices were gravity samplers and four were vacuum samplers. The gravity samplers were aluminum funnels 0.61 m (2 ft) in diameter and 0.30 m (1 ft) high, packed with glass wool.[17] A 1.91-cm (0.75-in.) PVC pipe was connected to the bottom of each funnel to transport the sample by gravity to the sampling shaft. The vacuum samplers were porous cup lysimeters. Two of these lysimeters were low rate (6.5 ml/min at 1 atm pressure) and two were high rate (8.0 to 9.0 ml/min at 1 atm). PVC pipe extended from each lysimeter to a 20.3-cm- (8-in.-) diam pit at grade, where tubing connections were provided for connection to the vacuum source and for sample collection. The vacuum source was a 1.91-cm (0.75-in.) PVC line extending from each lysimeter to the sampling shaft where the vacuum system was installed.

Sampling Point 4 consisted of six sampling devices installed in unsaturated soil 1.22 m (4 ft) below the leaching line closest to the distribution box. These devices were identical to those used for Sampling Point 3.

Three sampling devices, referred to as Sampling Point 5, were installed in unsaturated soil under the lip of the shield. Each device consisted of a 1.22-m (4-ft) length of 10.16-cm- (4-in.-) diam PVC pipe capped at both ends. A slot was formed along the length of each pipe by cutting off a 2.54-cm- (1-in.-) high segment from one end of the pipe to the other. The devices were installed, slot up, under the lip of the shield, each sampler directly under a leaching lne, and filled with gravel. A 1.91-cm (0.75-in.) PVC pipe was connected to each sampler to transport the collected sample by gravity to the sampling shaft. These devices yielded very little sample. Perhaps glass wool would have been a better fill material.

Four siphon-type samplers, referred to as Sampling Points 6, 7, 8, and 9 were installed in the pan. Each sampler consisted of a 0.91-m (3-ft) length of 2.54-cm (1-in.) perforated PVC pipe connected to the sampling shaft by 1.91-cm (0.75-in.) PVC piping. Rather than cutting through the pan wall to extend the 1.91-cm (0.75-in.) piping directly to the sampling shaft and thus introduce a possible source of pan leakage, the piping was brought up and over the side of the pan, and then extended down to the sampling shaft. A slight vacuum was used to start the siphon action for sample withdrawal.

Sampling Point 10 consisted of a valved tap on the flexible tubing connected to the pan outlet pipe in the sampling shaft.

To increase flexibility of the system, a 7.62-cm-(3-in.-) diam well was installed at the head of the pan to provide a point to take samples, feed liquids or chemicals, or check the pan liquid depth. This well was Sampling Point 5W.

OPERATION OF FACILITIES

The system became operational in December 1975. At that time the septic tank was filled with scavenger waste and daily feed of raw wastewater to the septic tank was begun; the impermeable pan had been filled with potable water during construction to test for water tightness. Through pump timer control and adjustment of valving on the pump discharge pipes, the pump discharged 0.114 m^3/h (30 gal/h) of wastewater to the septic tank from 6 a.m. to 7 p.m., inclusive. This pumping schedule permitted introduction of about 1.5 m^3/d (400 gal/day) of wastewater to approximate the flow from an individual household. During system operation the site was visited at least twice a week for monitoring and servicing, taking of samples, and feeding with methanol.

Flow. Flow into the system and flow discharged from the system were measured. Raw wastewater was pumped into the septic tank at 1.514 m^3/d (400 gal/day). When occasional pumping failure is taken into account, an average of 1.287 m^3/d (340 gal/day) was pumped into the system from July 1976 to June 1977, the period of flow record. During this same period, the average flow rate at the pan discharge (Sampling Point 10) was 0.341 m^3/d (90 gal/day) or about 25% of the inflow rate.

At the time of writing of this paper, the cause of the loss indicated by the flow data had not yet been determined. Several possibilities have been considered, including leakage from the septic tank or pan, and evapotranspiration. It has been confirmed that the septic tank and pan were not leaking and were not responsible for this flow difference. The concentration of chlorides did not vary appreciably from sampling point to sampling point or with time, indicating that evapotranspiration was not responsible for loss of flow. Therefore, other mechanisms must be responsible.

One mechanism that has been suggested is referred to as the "wick effect." Because of the different hydraulic properties of fine-grained soil and coarse-grained soil under unsaturated conditions, water percolating downward through a fine soil will, at the interface between the fine soil and an underlying layer of coarse soil, move laterally through the fine soil rather than transfer to the coarse soil.[18] Such a condition exists at the lower edge of the impermeable shield, where fine native soil is underlain by coarser pea gravel within the pan; flow loss may have occurred at this interface.

Sampling. Full-scale sampling began in March 1976. Samples were collected and analyzed on a weekly basis. Three-day composite samples were taken from Samplers 3 (gravity), 3 (vacuum), 4 (gravity), 4 (vacuum), and 5, and grab samples were taken at all other sampling points. On the average, 115 analyses were performed each week from March 1976 to May 1977. During the study, the parameters listed in Table I were measured. Because of field conditions or laboratory constraints beyond the control of the research team, not all analyses were performed at every point for the entire study period. Nitrogen analyses were performed by the Suffolk County Water Authority laboratory and all other analyses were performed by the Suffolk County Department of Health Services laboratory. Both laboratories are approved by the New York State Department of Health.

The following method was used to obtain the composite samples. Three days before the weekly sampling, Samplers 3, 4, and 5 were drained. The gravity sampler collection tubes in the sampling shaft then were inserted into collection bottles and the vacuum system was turned on and adjusted to maintain 0.9 atm absolute pressure in the vacuum samplers. Data for Sampling Point 3 are a composite of data from all Point 3 samplers; similarly, data for Sampling Points 4 and 5 are composites of data from all samplers at each of these respective points.

Methanol. Methanol was added to the pan daily starting in October 1976. Methanol feed was related to the nitrate, nitrite, and dissolved oxygen concentrations of the wastewater entering the pan, according to the following equation:

$$C_{CH_3OH} = 2.47\ C_{NO_3} + 1.53\ C_{NO_2} + 0.87\ C_{O_2}$$

where:

C_{CH_3OH} = required methanol concentration, mg/l,

C_{NO_3} = nitrate-nitrogen, mg/l,

C_{NO_2} = nitrite-nitrogen, mg/l,

C_{O_2} = dissolved oxygen, mg/l.

TABLE I. Parameters Measured, March 1976 through May 1977

Basic Group	Parameters
Nitrogen (N)	Ammonia (NH_3)
	Nitrite (NO_2^-)
	Nitrate (NO_3^-)
	Total Kjeldahl nitrogen
pH and alkalinity	pH
	Total alkalinity
	Phenolphthalein alkalinity
Oxygen and demand	Dissolved oxygen
	Biochemical oxygen demand
	Chemical oxygen demand
Solids	Suspended solids
	Volatile suspended solids
	Dissolved solids
	Specific conductance
Phosphorus	Total phosphate
	Orthophosphate
Coliform	Total coliform
	Fecal coliform
Metals	Iron
	Manganese
	Copper
	Zinc
	Sodium
Other	Temperature
	Chloride
	Methylene-blue-active substance

Daily flow discharged from the pan was used to determine the volume of methanol feed necessary to meet the required methanol concentration. Since the volume of methanol needed each day was quite small, one day's methanol feed was diluted with 0.0038 m^3 (1 gal) of water in each of the methanol feed system containers to provide uniform distribution of the methanol through the manifold.

Starting January 5, 1977, the methanol feed was increased to 1.5 times that required by the equation above. Starting February 23, 1977, and continuing through to the end of May 1977 the methanol feed was increased to twice that required by the equation above.

RESULTS AND DISCUSSION

This facility was intended to have three distinct treatment areas: the septic tank, unsaturated soil below the leaching field, and a saturated zone in the pan. The septic tank was intended to provide separation of settleable and floatable solids from the wastewater. The unsaturated soil was intended to provide aerobic conditions favorable to nitrification of ammonia. The saturated zone was intended to provide anaerobic conditions favorable to denitrification of nitrate. These three areas are discussed separately in this section.

Data presented in the tables in this section represent monthly averages of data from weekly samples. Because these monthly figures represent results obtained from once-a-week grab sambles, the value of each monthly figure by itself should not be considered conclusive. The data can be interpreted with more confidence if trends over several months are considered.

The following methods were used for sample analysis:

1. Ammonia nitrogen---automated colorimetric phenate method (Storet no. 00610) of the EPA.

2. Nitrite nitrogen---automated cadmium reduction method (Storet no. 00630) of the EPA.

3. Nitrate nitrogen--see nitrite nitrogen.

4. Total Kjeldahl nitrogen---digestion by the organic nitrogen method in "Standard Methods;"[13] ammonia nitrogen by the automated colorimetric phenate method (Storet no. 00610) of the EPA.

5. Total alkalinity---alkalinity in "Standard Methods."[13]

6. pH---pH value in "Standard Methods."[13]

7. Dissolved oxygen---azide modification in "Standard Methods."[13]

8. Total coliform---standard total coliform MPN tests in "Standard Methods."[13]

9. Fecal coliform---fecal coliform MPN procedure in "Standard Methods."[13]

Septic tank. Analyses of weekly samples show that the quality of raw wastewater was variable and that the septic tank equalized the quality of incoming wastewater and thereby produced a more uniform effluent. Because the quality of raw wastewater was variable, to assess the overall effectiveness of the septic tank as a treatment unit it is useful to examine averages of data collected over the entire study period. Such averages for data from Sampling Points 1 and 2 are presented in Table II.

Table II shows that there was a decrease in most parameters as the wastewater passed through the septic tank. Of prime interest are the nitrogen data. Inorganic nitrogen concentration decreased from an average of 51.1 mg/l in the raw wastewater to 40.6 mg/l in the septic tank effluent, a reduction of about 20%.

Total Kjeldahl nitrogen analyses have been performed on several samples of raw wastewater and septic tank effluent. These analyses indicate that approximately 33% of the total nitrogen in the raw wastewater and in the septic tank effluent is in the organic state.

Total and fecal coliform in raw wastewater and septic tank effluent were consistently above 2,400/100 ml.

Unsaturated soil. Monthly averages of ammonia and nitrate data from sampling points within the unsaturated soil are presented in Table III. This table shows that up to June 1976, ammonia concentrations were decreasing and nitrate concentrations were increasing in samples taken from all three sampling points. Samples from these points during this period also exhibited decreasing pH and bicarbonate alkalinity concentrations. These data indicate that the nitrification process was developing within the soil during this period. In the months following June 1976 the ammonia and nitrate concentrations remained somewhat constant, with the exception of the 0.61 m (2 ft) sampler (Point 3), which for several months continued to exhibit development of nitrification. Nitrite nitrogen concentrations throughout the unsaturated soil averaged less than 0.1 mg/l.

A rise in ammonia concentrations and a drop in nitrate concentrations seem to have accompanied lower temperatures at these points during winter months. Lack of complete temperature data caused by freezing at these points during the winter, and variability of inorganic-nitrogen concentration in septic tank effluent make it difficult to assess accurately the effect of cold weather on nitrification. However, it seems that nitrification occurring deeper in the soil was less affected by cold temperatures than nitrification occurring closer to the ground surface. In spite of an abnormally cold winter for this area, most of the ammonia entering the leaching field converted to nitrate within 0.61 m (2 ft) of soil travel. From the ammonia and nitrate data in the period following winter, it seems that the nitrification process recovers from its reduced winter rate, but does so slowly.

During the 12-month period ending May 1977, the average inorganic nitrogen concentrations for samples from 0.61 and 1.22 m (2 and 4 ft) below the leaching field were 35.2 and 34.2 mg/l, respectively. Total Kjeldahl analyses performed on several occasions indicate an approximate 80% decrease of organic nitrogen between the septic tank effluent and the unsaturated soil leachate samples. From this information and nitrogen data from the previous discussion on septic tank effluent, the total concentrations for the 12-month period ending May 1977 are calculated to be as follows:

Septic tank effluent	61.3 mg/l total nitrogen
0.61 m (2ft) sampler	39.2 mg/l total nitrogen
1.22 m (4 ft) sampler	38.2 mg/l total nitrogen

Other investigators have suggested mechanisms of nitrogen removal in the soil. Such mechanisms may be responsible for the 36% to 38% decrease in nitrogen exhibited in the data above, and include biological denitrification, volatilization of ammonia, removal by vegetation, and loss by chemo-denitrification.[19]

Fecal coliforms in samples from unsaturated soil were consistently less than 3/100 ml and total coliforms were usually less than 3/100 ml, indicating that the soil is very effective in removing these organisms.

Saturated zone. Monthly averages of ammonia and nitrate data from sampling points within the saturated zone are presented in Table IV. Data available through August 1976 at all pan sampling points show increased nitrate and ammonia concentrations as leachate filled the pan, displacing the tap water with which the pan was initially filled. Then ammonia concentrations decreased and nitrate concentrations continued to increase as nitrification occurred. After August 1977, monthly

TABLE II. Raw Wastewater and Septic Tank Effluent Data

Parameter	Raw Wastewater	Septic Tank Effluent
NH_3-N (mg/l)[a]	50.1	41.0
NO_2^--N (mg/l)[a]	0.1	0.0
NO_3^--N (mg/l)[a]	0.2	0.1
Inorganic - N (mg/l)[a]	50.4	41.1
Organic - N (mg/l)[a, b]	24.8	20.2
Total - N (mg/l)[a, b]	75.2	61.3
Total Alkalinity (mg/l as $CaCO_3$)[c]	246	186
pH [c]	7.5	6.9

---

a. June 1976 to May 1977
b. Calculated, based on organic N = 33% total N
c. March 1976 to September 1976

TABLE III. Data for Unsaturated Soil

Monthly Average
(mg/l)

	Sampling Point 3		Sampling Point 4		Sampling Point 5	
Time Period	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N
1976						
March	32.2	1.0	30.1	0.1	17.4	0.2
April	35.0	10.1	43.3	3.4	22.5	2.3
May	17.6	41.6	24.5	32.4	21.5	16.9
June	2.6	35.1	0.5	36.7	5.9	41.4
July	3.6	33.2	0.6	33.7	0.9	49.6
August	2.9	40.5	0.1	40.1	0.2	44.1
September	0.9	46.9	0.5	45.3	0.9	30.0
October	0.2	34.8	0.5	34.7	-	-
November	2.4	28.9	0.6	30.2	-	-
December	9.6	17.6	0.2	29.4	-	-
1977						
January	7.4	26.2	2.1	19.9	-	-
February	-	-	6.1	28.0	-	-
March	3.4	24.5	5.0	30.4	3.6	30.5
April	2.8	31.2	2.4	28.3	-	-
May	2.9	28.8	3.0	31.5	-	-
Avg., June 1976-May 1977	3.5	31.6	1.8	32.4	-	-

averages of ammonia-nitrogen concentrations were, with minor exceptions, consistently below 5 mg/l. Nitrite-nitrogen concentrations throughout the pan averaged less than 0.1 mg/l.

Data for September show decreased nitrate concentrations throughout most of the pan, indicating, perhaps, the occurrence of natural denitrification. Weekly data showing this decreasing trend were not yet available in the beginning of October when methanol addition was begun. From November 1976 through May 1977, samples from Points 5W, 6, and 8 exhibited nitrate concentrations consistently below the limit set for drinking water, and also exhibited pH and alkalinity concentrations generally above premethanol concentrations. These data indicate that denitrification was occurring at these points.

Samples from Points 7, 9, and 10 did not exhibit low nitrate or high alkalinity, indicating that denitrification was not occurring at these points. Dissolved oxygen was measured at Sampling Points 6 through 10. These data show low DO (less than 0.1 mg/l) in samples from Points 6 and 8, which are on the bottom of the pan and at which denitrification occurred, and high DO in samples from Point 7 (about 0.5 mg/l) and Point 9 (over 3.0 mg/l) which are mid-depth in the pan and at which denitrification did not occur. Samples from Point 10, representing a cross section of the pan depth, also exhibited high DO (between 1.0 and 2.0 mg/l). Apparently, oxygen transfer took place at the water surface in the pan and prevented anaerobic conditions, necessary for denitrification, from developing in the upper portions of the pan.

SUPPLEMENTARY WORK

A virus study and an organics and metal study, conducted as part of a Section 208 investigation under Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) for the Nassau-Suffolk Regional Planning Board, included wastewater sampling at this facility. Complete data from these studies will be included in the final 208 report.

Virus. One of the studies, by J.M. Vaughn and E.F. Landry, Department of Applied Science, Brookhaven National Laboratory, assessed the occurrence of human viruses in Long Island aquatic systems. One raw wastewater sample and one effluent sample were taken each month, over an 11-month period beginning July 1976, and analyzed for coliform and a variety of members of the human enterovirus group. The number of samples taken was not sufficient to be considered conclusive. However, from the data available and within the constraints of sampling procedures and analytical techniques, it seems that the facility is capable of excellent virus removal.

Organics and metals. The other study for which samples were taken was the Organic and Metal Sampling Program. Under this study, samples were analyzed for 9 volatile organics, 11 nonvolatile organics, and 8 metals. Sufficient data were not available for comment when this report was prepared.

CONCLUSIONS

The septic tank reduced the inorganic nitrogen (ammonia, nitrate, and nitrite) concentration of the raw wastewater about 20%. A similar reduction in organic nitrogen concentration occurred.

It took approximately 6 months to achieve steady-state nitrification within the soil.

Approximately 36% of the total nitrogen applied to the soil appears to have been removed within 0.6 m (2 ft) of travel through the soil.

Of the total nitrogen present in samples from the unsaturated soil, approximately 80% occurred as nitrate at the 0.6-m (2-ft) sampler, and 85% occurred as nitrate at the 1.2-m (4-ft) sampler. Alkalinity in the wastewater applied to the soil was depleted rapidly, and pH was depressed, in the aerobic soil where nitrification occurred.

Denitrification established itself within 1 month of commencement of methanol addition.

Denitrification occurred within approximately the first metre of travel through the pan, where nitrate-nitrogen concentrations as low as 0.1 mg/l frequently occurred. Alkalinity increased and pH rose in the anaerobic portions of the pan where denitrification occurred.

Denitrification did not occur in the upper layers of the pan, where DO concentrations were high.

Excellent coliform removal was achieved within 0.6 m (2 ft) of wastewater travel through the soil.

The study demonstrated that nitrification of septic tank effluent occurs within 0.6 to 1.2 m (2 to 4 ft) of vertical travel in a typical Long Island soil. It was further demonstrated that denitrification of nitrified septic tank effluent can be achieved under controlled conditions in the ground below a conventional leaching field. In order to develop a practical denitrification system for household wastewater disposal, the following work will be necessary:

TABLE IV. Data for Saturated Soil

Time Period	Monthly Average (mg/l)											
	Sampling Point 5W		Sampling Point 6		Sampling Point 7		Sampling Point 8		Sampling Point 9		Sampling Point 10	
	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N	NH_3-N	NO_3^--N
1976												
March	-	-	-	-	-	-	-	-	-	-	3.7	0.2
April	-	-	12.0	0.6	4.8	0.3	8.8	0.0	10.3	15.2	11.8	2.8
May	-	-	19.3	3.4	8.8	3.8	14.5	0.7	11.9	38.1	12.1	14.1
June	-	-	12.7	8.7	11.7	15.7	12.7	43.3	3.7	45.4	11.1	27.0
July	5.8	23.5	10.9	6.4	9.7	20.1	9.6	34.9	2.2	40.6	4.8	30.1
August	1.8	39.5	4.9	33.8	5.7	28.7	4.6	37.3	1.6	41.0	3.9	30.6
September	1.2	28.4	4.6	33.9	3.0	19.0	4.0	27.9	0.4	32.8	3.5	22.8
October	1.1	14.1	6.4	17.3	3.1	13.5	2.7	15.4	0.7	22.0	2.9	24.2
November	1.3	1.8	4.0	3.4	3.9	8.3	2.2	8.2	1.1	21.7	2.3	22.8
December	1.7	8.4	1.6	0.1	3.1	16.7	3.3	1.5	0.3	30.7	1.1	22.0
1977												
January	4.0	6.2	0.9	0.4	2.2	19.7	2.5	2.1	0.0	28.2	0.6	20.8
February	2.1	3.0	1.1	0.6	2.3	19.0	2.5	2.8	0.0	25.9	1.2	24.4
March	0.9	0.1	1.4	0.1	2.5	16.9	2.6	2.5	0.1	27.4	2.4	19.9
April	0.8	0.2	1.5	0.1	2.0	26.9	2.5	3.9	0.8	28.9	3.9	27.3
May	0.6	0.1	1.0	0.2	1.5	29.6	2.2	3.0	0.0	28.8	1.9	28.4
Average November 1976-May 1977	1.6	2.8	1.6	0.7	2.5	19.6	2.6	3.4	0.3	27.4	1.9	23.7

- Continue operating the existing system and assess the response to varied loading conditions such as increased or decreased wastewater flow, addition of chemicals to the septic tank, and discontinued methanol feed. Further investigate the fate of nitrogen and flow to acquire a more complete understanding of the processes at work in the soil. This work is presently under way.

- Investigate alternate carbon sources, including the gray water such as sink and laundry waste, and long-lasting substrates.

- Construct a second-generation experimental disposal system using leaching pools in lieu of a leaching field, a nitrification zone of shorter depth, and a more efficient denitrification zone.

ACKNOWLEDGMENTS

Credits. The authors wish to thank August Guerrera of the Suffolk County Water Authority and Brookhaven National Laboratory personnel for their invaluable assistance throughout the study. The authors are grateful to Keith Porter of Cornell University for his extensive technical review of the draft report. Particular thanks are due William Slade, a Sanitarian with the Suffolk County Department of Health Services, for his assistance in operating the facility.

Authors. Aldo Andreoli is Principal Public Health Engineer, Bureau of Environmental Health Services, Suffolk County Department of Health Services, Hauppauge, N.Y., Nicholas Bartilucci and Robert Forgione are, respectively, Principal and Project Engineer, Dvirka and Bartilucci, Woodbury, N.Y., and Royal Reynolds is Assistant Public Health Engineer, Bureau of Environmental Health Services, Suffolk County Department of Health Services, Riverhead, N.Y.

REFERENCES

1. Cohen, P., et al., "An Atlas of Long Island's Water Resources." U.S. Geological Survey and New York State Water Resources Commission, N.Y. (1968).

2. Sgambat, J.P., and Miller, D.W., "Water-Supply Philosophies and 208 Planning." Geraghty & Miller Inc., Port Washington, N.Y. (preliminary draft, Apr. 1977).

3. "The Long Island Ground Water Pollution Study." New York State Department of Health, Long Island, N.Y. (1969).

4. Porter, K.S., and Zaeh, D., "Assessment of Nitrogen Sources -- A Summary." Cornell Univ., Ithaca, N.Y. (first draft Aug. 1, 1977).

5. Perlmutter, N.M., and Koch, E., "Preliminary Hydrogeologic Appraisal of Nitrate in Groundwater and Streams, Southern Nassau County, New York." U.S. Geol. Surv. Res. (1972).

6. Myott, D., Public Health Engineer, Bureau of Water Resources, Nassau County Department of Health, Personal communication (July 14 and 15, 1977).

7. "Denitrification by Anaerobic Filters and Ponds." 13030 ELY 04/71-8, U.S. EPA, Office of Research and Monitoring, Washington, D.C. (Apr. 1971).

8. Erickson, A.E., et al., "A Barriered Landscape Water Renovation System for Removing Phosphate and Nitrogen from Liquid Feedlot Waste." Proc. Int. Symp. Livestock Wastes, Amer. Soc. Agri. Engr., St. Joseph, Mich., p. 232.

9. "Nitrification and Denitrification Facilities." U.S. EPA, Technology Transfer, Washington, D.C. (Aug. 1973).

10. "Process Design Manual for Nitrogen Control." U.S. EPA, Technology Transfer, Washington, D.C. (Oct. 1975).

11. Sawyer, C.N., and McCarty, P.L., "Chemistry for Sanitary Engineers." 2nd Ed., McGraw-Hill Book Co., New York, N.Y. (1967).

12. Erickson, A.E., et al., "Soil Modification for Denitrification and Phosphate Reduction of Feedlot Waste." EPA-660/2-74-057, U.S. EPA, Off. Research and Development, Washington, D.C. (June 1974).

13. "Standard Methods for the Examination of Water and Wastewater." 14th Ed., Amer. Pub. Health Assn., Washington, D.C. (1975).

14. Brandes, M., et al., "Environmental Study on Removal of Pollutants from Domestic Sewage by Underdrained Soil Filters." West Central Region, Ontario Ministry of the Environment, Toronto, Ontario, Canada (1973).

15. Jeris, J.S., and Owens, R.W., "Pilot Scale High Rate Biological Denitrification at Nassau County, N.Y." Ecolotrol, Inc., Bethpage, N.Y. (Jan. 1974).

16. English, J.N., et al., "Denitrification in Granular Carbon and Sand Columns." Jour. Water Poll. Control Fed., 46, 28 (1974).

17. "Study of Reutilization of Wastewater Recycled Through Groundwater." Vol. 1, 16060 DDZ 07/71, U.S. EPA, Off. Research and Monitoring, Washington, D.C. (July 1971).

18. Frind, E.O., et al., "Application of Unsaturated Flow Properties in the Design of Geologic Environments for Radioactive Waste Storage Facilities." Dept. Earth Sciences, Univ. Waterloo, Ontario, Canada (July 1976).

19. Lance, J.C., "Nitrogen Removal by Soil Mechanisms." Jour. Water Poll. Control Fed., 44, 1352 (1972).

OZONE APPLICATIONS: A WATER REUSE REVIEW

Harvey M. Rosen
Union Carbide Corporation, Ozonation Systems
Tonawanda, NY 14150

INTRODUCTION

Water reuse in most cases will be indirect, i.e., treated wastewater will be ground injected, stored in surface water reservoirs, or mixed with other "pure" water supplies. After some variable time, this water will then undergo a second treatment stage, usually called water treatment as opposed to the wastewater treatment it received prior to its initial discharge. Often a middle stage of treatment, advanced waste treatment (AWT), is added to the end of more traditional wastewater treatment when reuse is in mind.

Until a relatively few years ago the solution to pollution was dilution, and the same water bodies were often used for wastewater discharges and water supplies, with very little thought or planning about the relationships between the two. This, of course, is changing as the scarcity and value of water resources are realized. Also, the definition of potable water purity has changed and continues to evolve as we recognize the potential dangers of and are more easily able to identify and quantify micropollutants.

Potable reuse is not new. For many years the rivers of Europe were its sewers, and water withdrawn from these sources required a much higher degree of treatment before use as compared with the much more pristine nature of water sources in the U.S. The use of ozone for wastewater reuse is well established in the field, since the commercial history of ozonation to treat potable water dates back to Europe in the early 20th century.

Ozone is currently being used widely outside the U.S. (over 1100 water treatment plants in operation) and is rapidly gaining acceptance here in treating both water and wastewater. Ozone has been demonstrated to have a place at both ends and the middle (AWT) of a potable reuse scheme.

APPLICATIONS FOR OZONE

As reuse schemes develop and become more direct, the differentiation between treated wastewater and raw potable water begins to disappear. This distinction becomes even less meaningful as the use of ozone for polishing or as part of AWT is considered, since some products of AWT are of higher quality in some areas than raw water sources in others. Thus, for the purposes of limiting discussion primarily to reuse for potable purposes, no differentiation will be made between wastewater and water and, unless otherwise stated, the term "water" will be used generically.

Ozone is the strongest oxidant available for water treatment. In its role as an oxidant it is also the strongest practical disinfectant known with respect to all types of microorganisms [1]. Thus the application of ozone to water and the resulting purification will generally fall into two categories, oxidation and disinfection. In addition, the process of ozonation can lead to improvements in water quality not necessarily connected with either, but they will be pointed out as special cases.

While the following discussion will address the advantages of ozone and will demonstrate its many uses through data and references, ozonation like any unit process does not stand alone, but is part of a multiple unit process treatment scheme to achieve a cost-effective treatment objective. Also, each water is different and, although discussions tend to generalize the utility of any unit process to achieve the cost effective objective when combined with the other possible unit processes, that utility will vary. As a result, for a particular water, ozonation will be only one method of treatment among and in combination with many others that may be necessary.

The capabilities of ozone will be reviewed, so that when a problem arises, the problem-solver can, at least in a preliminary searching, evaluate its potential to help him. The next step necessary is then to try ozone in a treatability or pilot plant situation, in combination with the other unit processes under consideration to determine its true utility for the situation at hand. Table 1 lists some of the possible reuse applications for ozone.

Disinfection

The difference between water and wastewater disinfection by ozone is only a matter of degree. The disinfection of wastewater requires more ozone because of the demands created by the other materials present which compete for the ozone. Thus, the quality of the wastewater which exhibits a decreasing ozone demand with increasing quality is the prime determinant of the required amount of ozone to achieve disinfection. Other determinants are the level of disinfection desired and the organisms to be inactivated.

Table 2 [1] demonstrates the relative efficiency of ozone as compared to other disinfectants on particular microorganism types, and the relative efficiencies of the various disinfectants to a particular class of microorganisms on the basis of a dose-time relationship. Figures 1-7 and Tables 3 and 4 present ozone disinfection data for various quality waters for various indicator organisms under varying experimental conditions. Since it is obviously impossible to cover all the conditions of each experiment, only a few general points will be made in evaluating the data.

The first point is extremely significant in evaluating all ozonation data. It is that, in most cases, unless otherwise specified, applied ozone dose is reported as opposed to utilized, absorbed or dissolved ozone. Since there are rarely enough data or experimental conditions reported, it can only be surmised that the ozone required in an efficient contactor would be less, in many cases significantly less, than the applied dose reported to achieve some specified level of treatment. This problem and others in designing ozonation experiments and interpreting data are discussed in a recent paper [2]. Other significant points in interpreting data are the analytical techniques used for ozone determination, contactor design, pH, similarity of conditions to real water conditions, etc. Too much is often concluded from too few data that are often experimental artifacts rather than practical, meaningful results. With this general caveat, which will not be repeated in the discussion for each of the exhibits, the previously mentioned disinfection data will be discussed with respect to a few salient points.

Figure 1 [3]: A continuous 30-day pilot study shows reliability and reproducibility of disinfection at 5 mg/l, 1.5 minute contact time to achieve a monthly geometric mean of 100 fecal coliforms per 100 ml. This level of disinfection is often not sufficient, especially when considering reuse. However, the data presented will be an indication of what might be done for reuse, since the data using ozone in potential reuse situations are so sparse. The only difference between the kinds of data presented and what could be expected from a situation with the same quality effluent but for a higher level of disinfection, would be a higher utilized ozone dose requirement for the higher disinfection level. In many situations, prior to ozone disinfection the water will have undergone AWT, and the dose to achieve any disinfection level will be reduced when compared with this unfiltered secondary effluent.

Figure 2 [4]: This figure shows the effect of ozone dose on fecal coliforms for an unfiltered secondary effluent. Ten fecal coliforms per 100 ml requires 10 mg/l ozone.

Figure 3 [4]: This figure shows the "same" effluent as in Figure 2 but, because suspended solids and COD have been reduced by filtration, the same level of disinfection requires only 6 mg/l.

Figure 4 [4]: In this figure, on the "same" effluent as in Figure 3, it can be seen that 8 mg/l ozone are required for approximately 10 total coliforms as compared with 6 mg/l for fecal coliforms only.

Figure 5 [5]: In this figure, a different secondary effluent is examined with respect to fecal coliforms. Approximately 6 mg/l are required to achieve a count of approximately 1 per 100 ml with a somewhat lower quality effluent than that in Figure 3. This figure also shows ozone disinfection is independent of contactor type, for the types studied, as long as each contactor has been optimized.

Figure 6 [6]: It can be seen here that as one goes from "dirty" to "clean" water, the efficiency of ozonation increases, so that it requires <0.5 mg/l to achieve a minimum standard plate count at 48 hours.

Figure 7 [7]: Figure 7 shows the relative resistance of a number of viruses to ozone. These data would tend to indicate that Echo-12 might be a good indicator organism for viral inactivation by ozone.

Table 3 [8]: Table 3 reports pilot plant data for coliforms and viruses in a 10 gpm pilot plant with 1.5 minute contact time. Again, the efficiency of ozone is demonstrated. Seventy-six million pfu/100 gal of Coxsackie B-3 are inactivated at a 1.1 mg/l applied dose. One hundred fifty gallon samples of ozonated water were concentrated before viral examination and showed no survivors. The Coxsackie B-3 virus was selected for pilot plant studies over Polio II because in batch studies it was found that Coxsackie B-3 was more resistant to ozone inactivation. These data are in conflict with those in Figure 7 relative to the resistance of different viruses to ozone, pointing out the need for more definitive studies.

Table 4 [3]: These data are for treated water with reclamation in mind. The effects of a constant 5 mg/l dose in a 25 gpm pilot plant on various stages of tertiary-treated wastewater effluent are presented. With increasing effluent quality, total bacteria and seeded bacterial viruses begin to be completely eliminated.

All the above data show that ozone disinfection efficiency increases as the effluent quality increases, and that reasonable doses of ozone are required for high effluent disinfection levels, both for bacteria and viruses. The data also present some conflicting specifics which only indicate what is already well known, independent of disinfectant - that the water in question must be examined using the treatment considered, for confidence in resulting quality.

Organic Removal by Direct Oxidation

The second primary use of ozone is chemical oxidation. In this area, ozone is such a strong oxidant that there are essentially no organic compounds that, from the point of view of thermodynamics, it cannot oxidize to CO_2 and H_2O. However, ozone oxidation at the low doses usually employed in water treatment is incomplete, having little effect on TOC, while at higher doses, usually considered too costly, major TOC reductions can be realized.

Smaller removals of TOC than would be expected from thermodynamic arguments result because, under the usual design conditions for ozonation, dissolved ozone concentrations are low and the kinetics of the organic oxidation reactions limit the utility of ozone to reduce the concentration of TOC. However, as the cost of alternatives increases, so does the cost-effective potential of ozone, especially as a polishing agent where removal of small amounts of TOC is what is required of ozone for reuse situations.

The most definitive study in this area was performed several years ago in the EPA Washington, DC, Blue Plains pilot plant [9]. One of the conclusions of the study which examined a variety of effluents was that a 50% COD removal (40 to 20 mg/l) could be achieved at a 1973 cost of $0.10/1000 gal for a 50 mgd plant. Another conclusion was that, "There appear to be no technical limits on the COD removal." The study further concludes that the rate of COD reduction is increased with increasing pH and increased dissolved ozone concentration.

The study then goes on to report that high ozone dose tertiary treatment results in practically bacteria-free water, and turbidity reductions are significant. Also, the study was performed under non-optimum conditions and savings could be expected with technology improvements, a number of which are here today compared to the 1971-1972 work. The efficiency of COD removal generally ranged from 3 parts of COD per part of ozone for high COD concentrations, to one part of COD per 3 parts of ozone at low concentrations. Since the study was extensive, only a brief description is presented, and if further details are required the original reference should be examined.

Figures 8, 9 and 10, taken from reference 9, show some of the results discussed above. Figures 11 and 12 [3] show the reductions of color and turbidity that accompanied the disinfection results shown in Figure 1.

Table 5 [10] presents data from a study conducted at the Dallas Water Reclamation Research Center. Liquid flow was held constant at 25 gpm at a 1.5 minute contact time and an applied ozone dose at 5 mg/l. Ozone concentration in the gas phase was changed to study the effects of gas to liquid ratio on the contactor performance, so that many runs did not result in high ozone utilization efficiencies. However, the data are an indication of how ozone may be used to polish an effluent for reuse.

At the other end of the spectrum, Figures 13 and 14 [11] show how a groundwater quality was improved by ozonation. These data as well as the earlier disinfection data show that in considering reuse, direct ozone oxidation has possibilities in both the wastewater and water treatment applications that complete the loop of reclamation and reuse.

Suspended Solids Removal by Flotation/Oxidation

Foulds [12, 13, 14] was the first to report the phenomenon of flotation when treating a secondary effluent with ozone. Some effluents, especially those containing specific organics which, when oxidized, cause froth, will produce a rigid foam which floats to the surface of an ozone contactor and can be removed for disposal by conventional (e.g., water spray) means.

Thus there is a physical removal of pollutants through the froth flotation mechanism, as well as any pollutant removal accompanied by direct oxidation. The froth flotation mechanism appears to be enhanced by small amounts of iron and/or manganese in the wastewater which, when oxidized by ozone, create pin floc nuclei around which the froth can develop. It has been reported [15] that this flotation mechanism can remove significant amounts of many different wastewater constituents, especially when looked at as part of a tertiary polishing scheme. The froth flotation mechanism is not well understood and many effluents, even of relatively low quality, do not foam when ozonated.

This phenomenon, however, was piloted and found to be effective for the 5 mgd ozonation system currently in start-up in Chino Basin, CA, [16] where the treated wastewater is to be discharged to Chino Creek. From there it may ultimately find its way to surface and groundwaters in Orange County where reuse is likely. The constraints on discharged wastewater quality included minimal incremental increase in TDS, specifically chloride and sulfate, and potable water quality levels of turbidity, bacteria and viruses. As demonstrated in Table 6, the ozonation step went a long way in meeting the criteria of high effluent quality without excessive addition of the TDS that higher dose conventional alum coagulation would have added, and probably reduced the final chlorine dose to achieve the disinfection standards. An earlier report of this study [17] indicated that the treatment scheme employing ozone not only was effective in achieving standards, but the overall treatment process employing ozone was estimated to be less costly than more conventional treatment by 8 to 10%.

The "froth" referred to above is not the detergent foam often found as a result of wastewater aeration, but has a more rigid, handleable consistency. In fact, ozone has been shown in some cases to reduce the more traditional effluent foaming problems because of its ability to oxidize surfactants [18, 19].

Microflocculation

The removal of turbidity and soluble organics, or improvement in these removals, can be accomplished by another ozonation mechanism, usually termed microflocculation [20, 21]. In a sense ozone acts as a flocculent aid, often allowing reduction in coagulants by making them more efficient. This phenomenon has been reported often, especially at low ozone doses in water treatment applications. This is another of the under-utilized (because it is not well understood) ozone applications which may have potential in reuse schemes.

Specific Micropollutant Removal

While the gross removal of organics through the measurement of parameters such as COD and TOC has been discussed, it should not be forgotten that ozone can be very selective in its reactions toward particular types of organics. Much of this is covered in a recent review [22], but two examples will be presented here. The point of this discussion is that, while ozone may not be an efficient method for the removal of gross quantities of organics, it can often be selective for a particular compound or group of compounds which are difficult to remove by other processes.

In an excellent piece of research conducted ten years ago, Gardiner and Montgomery [19] reported on the treatment of settled trickling filter effluents. Table 7 shows the effect of ozone on pesticides added to this wastewater. When a mixture of phenols, including phenol, o-, m- and p-cresol and catechol, was added to the wastewater, an absorbed ozone dose of 25.6 mg/l reduced the total phenols concentration from 11.8 to 2.3 mg/l or an 81% reduction, at the same time that the wastewater COD was reduced from 76 to 62 mg/l.

Table 8 summarizes the work of Italian investigators [23] on the effect of ozone on polynuclear aromatic hydrocarbons (PAH) in river and finished drinking water.

Stripping for Volatile Organics Removal

The process of ozonation is one in which relatively large gas volumes are added to the water. At a nominal 1 wt.% concentration of ozone in the gas phase, 100 pounds of air are added for every pound of ozone. Thus, at a 5 ppm dose, 38.8 SCFM would be applied to each million gallons. As a result, certain volatiles will be stripped and removed from solution. The pesticide results reported above may be largely due to this phenomenon, since highly chlorinated organics are generally refractory to ozonation.

Another example of this kind of removal mechanism was demonstrated by Barnett and Trussell [24], during an ozonation study of Lake Casitas, CA, water where 2 mg/l applied ozone resulted in a concentration reduction of TOC from 8-9 mg/l to 3-4 mg/l. The only reasonable explanation for this observation is that stripping of volatiles which contribute to TOC accompanied any oxidation that occurred.

Again, the important conclusion to be drawn is not that ozonation should be used for aeration, but that as a natural consequence of the process there may be several removal mechanisms working together in a single process step to produce the desired quality product.

Pretreatment Prior to Biological Treatment or GAC

1. Biological Treatment. There has been a little work on the treatment of wastes prior to biological treatment in order to detoxify or make the waste less refractory so that biological treatment can be more efficient. This treatment has been considered both prior to biological treatment for removal of carbonaceous material as well as prior to nitrification. Since ozone is very often selective on a kinetic basis for biologically refractory material and tends to produce a higher BOD/COD before BOD is ultimately oxidized and TOC removal occurs, pretreatment with ozone can make a waste more amenable to biological treatment for removal of carbonaceous material. Also, since ozone can detoxify many of the materials which tend to inhibit nitrification, a similar pretreatment may help enhance biological ammonia oxidation. In this area both the oxidation and stripping action resulting from ozone treatment can be useful. Stuber [25] reported on the effect of ozone on the BOD/COD ratio of a wastewater containing a high industrial load. The results are shown in Figure 15. More recently, Yocum and coworkers [26] have studied this ozone application.

2. Ozone Enhanced Biologically Active Carbon (OEBAC). Biologically active carbon, commonly referred to as BAC, is one of the areas of high current interest in ozonation. Since essentially all granular activated carbon (GAC) systems support biological growth, the name is somewhat inappropriate; hence, the title of this section, ozone enhanced biologically active carbon or OEBAC. A good review of OEBAC was recently

published by Miller and Rice [27]. Because of the potential value of this process to both water and wastewater and perhaps especially AWT for reuse, it will be discussed here.

Table 9 lists the process concepts associated with OEBAC and is fairly self-explanatory. It should be pointed out that the enhancement observed in organic removal, when compared with the use of oxygen alone for enhancement of biological growth on the carbon, more than justifies the ozonation costs.

Table 10 lists the potential advantages of OEBAC. OEBAC has been studied and is operating for water treatment [28], wastewater treatment as part of a physical-chemical treatment scheme with no biological treatment [29], and for tertiary polishing of a biologically treated wastewater with reclamation in mind [30].

Figures 16 and 17 demonstrate some OEBAC results from a Swiss study [31]. Even though it appears from Figure 16 that the carbon is completely loaded in approximately 16 weeks, Figure 17 shows that effluent quality after 50 weeks is still comparable to that reported for new carbon.

Table 11 presents data from a French waterworks [28]. These data show the multiple effects of oxidation, stripping, microflocculation and OEBAC that can be achieved by ozonation.

Figure 18 shows the results of an OEBAC study conducted in Cleveland [29]. The Cleveland Westerly WWT plant is currently designing a 50 mgd facility based on the use of OEBAC following lime clarification, to achieve a 15 mg/l TSS, 15 mg/l BOD and approximately a 50 mg/l COD on a wastewater with a high industrial component which was difficult to treat biologically.

Figure 19 shows the pilot plant results for tertiary polishing of a lime-treated activated sludge effluent [30]. The COD level of less than 2 mg/l and TOC of less than 1 mg/l show the effectiveness of this treatment. Sufficient information regarding ozone dose or a comparison with GAC alone was not included. The short discussion and limited information presented, however, do suggest that OEBAC can be a powerful tool for water treatment and especially for effluent polishing prior to reuse.

Catalytic Ozonation

The last area of ozonation that requires some discussion because of its potential value in reclamation applications and because it has been studied a good deal is catalytic ozonation. Four categories, soluble chemical catalysis, insoluble chemical catalysis, ultrasonic catalysis, and ultraviolet (UV) catalysis, require mention.

1. Soluble Chemical Catalysis. While little is known about all the potential catalytic effects of water soluble catalysts, primarily because of their complexities, two cases are worth pointing out. The first is the catalytic effect of cupric ion on the oxidation of cyanides [32]. While it is not implied that this catalyst and pollutant would necessarily be

of specific use in reclamation for potable purposes, it is certainly important in industrial reuse and it does point out the type of catalytic effects possible which may affect the treatment of a specific water.

The second and more important catalytic effect comes as a result of production of hydroxyl radicals. When water is ozonated it is postulated that oxidation occurs both directly through molecular ozone [33, 34], and by hydroxyl radicals. The formation of these radicals can be enhanced by several techniques including raising the pH and, if this type of catalytic ozonation is understood and used to advantage, the cost-effectiveness of ozone oxidation may be improved.

2. Insoluble Chemical Catalysis. Chen et al [35] have reported on the effect of such catalysts as Raney nickel on the oxidation of specific compounds in water. While the catalytic effect is present, a very inexpensive catalyst that would not add greatly to sludge production or is easily recoverable will be required before the use of such catalysts can be justified in wastewater treatment.

3. Ultrasonic Catalysis. Ultrasonic catalysis is listed as a catalytic effect although in some instances an observed improvement in ozone oxidation efficiency may be due to the sonic emulsification of the wastes to produce good mixing and turbulence for ozone mass transfer and reduction of particle size of solids, rather than a true catalysis. Sierka [36], however, has shown (Figure 20) that sonification can apparently catalyze the ozone oxidation of specific compounds.

4. UV Catalysis. This is the area in which the most work on catalysis of aqueous ozonation reactions has been performed [37, 38, 39]. Table 12 presents some data showing this catalytic effect with respect to particular compounds refractory to traditional ozonation. Figure 21 shows data comparing ozone and UV ozonation on a potable water source for trihalomethane precursor oxidation [39].

SUMMARY AND DISCUSSION

Some of the applications discussed above result only in modification of organic molecules, such as in the case of low dose applications that oxidize trihalomethane precursors to reduce the formation of trihalomethanes upon ultimate chlorination [40, 41]. Other applications result in major TOC reductions. Direct ozone oxidation in some cases and OEBAC and UV ozonation have been shown to do this.

In all cases, ozonation results in improved water quality. One general benefit is high dissolved oxygen. Also, when ozone is evaluated it must be evaluated as part of a total unit process scheme to produce the required product quality, and the multiple effects of disinfection, TOC oxidation, possible solids removal, increase of DO, etc., that may result and how they may create savings in other unit processes require study.

Lastly, by necessity the data presented are incomplete, so it is difficult to evaluate cost-effectiveness. This is often because the data presented in the references are incomplete so cost-effectiveness cannot be judged. Furthermore, the variety of water and test conditions do not allow direct comparison or extension to other waters under different conditions.

Thus, the purpose of the foregoing is to suggest the possibilities available in the use of ozone in treating all quality waters from secondary effluents to high purity potable waters, and because of its proven versatility and advantages to suggest that the production of high quality water in any stage of the reuse cycle must consider ozone as a possible part of the treatment sequence, and to evaluate its usefulness through testing.

REUSE/RECLAMATION EXPERIENCE

The previous discussion makes it evident that there are a number of reuse systems which employ ozonation and many more under study. Table 13 lists some of these systems. Depending on the definition of reuse, many more systems where ozone is used for tertiary polishing might be added to the list. However, these systems were generally designed and designated as AWT systems but are not designated as reuse systems, although this may have been in mind.

While Table 13 is necessarily incomplete and all reclamation is not for direct potable reuse, the breadth of application gives some idea of the widespread use of ozone in this field.

A recent review of industrial reuse applications of ozone is available [48], as well as technical information on ozone system requirements [49-60].

REFERENCES

1. MORRIS, J.C. Aspects of the Quantitative Assessment of Germicidal Efficiency, Disinfection-Water and Wastewater (J. D. Johnson, editor), Ann Arbor Science Publishers, Inc., Ann Arbor, MI (1975).

2. BREITBACH, G.P. ET AL. Ozone Testing Programs: An Approach for Valid and Comparable Results. ACS 175th National Meeting, Anaheim, CA (March 1978).

3. ROSEN, H.M. ET AL. Economical Wastewater Disinfection with Ozone, Disinfection-Water and Wastewater (J.D. Johnson, editor), Ann Arbor Science Publishers, Inc., Ann Arbor, MI (1975).

4. ADAMS, R.W. & BENNETT, B.D. Recent Studies on Ozone for Water and Advanced Wastewater Treatment. International Ozone Institute Ozone Symp. & Expos., Los Angeles, CA (May 1978).

5. SCACCIA, C. & ROSEN, H.M. Ozone Contacting: What is the Answer? Int'l Ozone Institute Symp. on Advanced Ozone Techn., Toronto, Canada (Nov. 1977).

6. Interim Treatment Guide for the Control of Chloroform and Other Trihalomethanes, USEPA (June 1976).

7. SNYDER, J.E. & CHANG, P.W. Relative Resistance of Eight Human Enteric Viruses to Ozonization in Saugatucket River Water. Aquatic Applications of Ozone (W.J. Blogoslawski & R.G. Rice, editors), Int'l Ozone Institute, Inc., Syracuse, NY (1975).

8. KELLER, J.W. ET AL. Ozone Disinfection Pilot-Plant Studies at Laconia, NH. Jour. AWWA (Dec. 1975), p. 730.

9. WYNN, C.S. ET AL. Pilot Plant for Tertiary Treatment of Wastewater with Ozone. USEPA Office of Research and Monitoring, Report No. EPA-R2-73-146, Washington, DC (Jan. 1974).

10. PETRASEK, A.C. JR. ET AL. The Impact of Ozone on Water Quality for Various Wastewaters. Proc. 1st Int'l Symp. on Ozone for Water & Wastewater Treatment, Int'l Ozone Institute, Inc., Waterbury, CT (1975), p. 232.

11. Ozonation Test Report - Collier Carbon and Chemical Corporation. Union Carbide Corporation, unpubl. report (May 1976).

12. FOULDS, J.M. ET AL. Ozone Generated Froth for Sewage Treatment. Water & Sewage Works (March 1971), p. 80.

13. FOULDS, J.M. What's It All About? Algae. Water & Wastes Engineering (Aug. 1972), p. 45.

14. FOULDS, J.M. Ozone Generated Froth for Sewage Treatment. Proc. 1st Int'l Symp. on Ozone for Water & Wastewater Treatment, Int'l Ozone Institute, Inc., Waterbury, CT (1975), p. 254.

15. NEBEL, C. ET AL. Ozone Pilot Plant Disinfection of Combined Industrial and Municipal Secondary Effluents. Proc. 1st Int'l Symp. on Ozone for Water & Wastewater Treatment, Int'l Ozone Institute, Inc., Waterbury, CT (1975), p. 405.

16. TATE, C.H. Ozone in Advanced Wastewater Treatment - Chino Basin Municipal Water District Case Study. NV WPCA 13th Annual Conf., Las Vegas, NV (Dec. 1977).

17. TRUSSELL, R. ET AL. Ozone Found Effective in Tertiary Treatment. *Bull. Calif. WPCA* 12:3:75 (Jan. 1976).

18. EVANS, F.L. & RYCKMAN, D.W. Ozone Treatment of Waste Containing ABS. 18th Indus. Waste Conf., Purdue Univ., West Lafayette, IN (1963).

19. GARDINER, D.K. & MONTGOMERY, H.A.C. The Treatment of Sewage Effluents with Ozone. *Water & Waste Treatment* 12 (Sept./Oct. 1968).

20. KUHN, W. ET AL. Use of Ozone and Chlorine in Water Utilities in the Federal Republic of Germany. *Jour. AWWA* (June 1978), p. 326.

21. BARKER, M.R. & PALMER, D.J. Persistent Odour-Taste Removal Using Ozone and Microcoagulation. *Inst. of Water Engrs. & Scientists* 31:2:109 (Mar. 1977).

22. ROSEN, H.M. Ozonation: A Solution to Potential Carcinogens in Drinking Water. Proc. 2nd Int'l Symp. on Ozone Techn., Montreal, Canada, Int'l Ozone Institute, Syracuse, NY (1976), p. 143.

23. SFORZOLINI, G.S. ET AL. Decontamination of Waters Polluted by Polycyclic Aromatic Hydrocarbons. II-Action of Chlorine and of Ozone on the PAH Solubilized in Drinking Water and in River Water. *Igiene Moderna* 66:6:595 (1974).

24. BARNETT, R.H. & TRUSSELL, A.R. Organics in Domestic Water Supplies - The Lake Casitas Experience. Proc. Water Treatment Forum VII, CA/NV Section AWWA (April 1978).

25. STUBER, L.M. Tufted Carpet Dye Wastewater Treatment, Part III. *Indus. Wastes* (May/June 1975).

26. YOCUM, F.H. ET AL. Pretreatment of Industrial Wastes with Ozone. *AIChE Symp. Series* 74:178:217 (1978).

27. MILLER, G.W. & RICE, R.G. European Water Treatment Practices - The Promise of Biological Activated Carbon. Civil Engrg.-ASCE (Feb. 1978), p. 81.

28. RICE, R.G. ET AL. Rouen, France Water-Treatment Plant: Good Organics and Ammonia Removal with No Need to Regenerate Carbon Beds. Civil Engrg.-ASCE (May 1978), p. 76.

29. HANNA, Y.A. ET AL. Ozone as a Pre-Treatment Step for Physical Chemical Treatment. Int'l Ozone Institute Symp. on Advanced Ozone Techn., Toronto, Canada (Nov. 1977).

30. WACHS, A. ET AL. Organic Matter Removal from Effluents by Lime Treatment, Ozonation and Biologically Extended Activated Carbon Treatment. Int'l Ozone Inst. 3rd Int'l Symp. & World Congress, Paris, France (May 1977).

31. SCHALEKAMP, M. Use of Activated Carbon in the Treatment of Lake Water. Transl. of Reports on Special Problems of Water Techn. Vol. 9: Adsorption, EPA-600/9-76-030 (Dec. 1976).

32. BOLLYKY, L.J. Ozone Treatment of Cyanide-Bearing Plating Waste. Env. Protection Techn. Series, Report No. EPA-600/2-77-104, USEPA, Cincinnati, OH (June 1977).

33. HOIGNE, J. & BADER, H. The Role of Hydroxyl Radical Reactions in Ozonation Processes in Aqueous Solutions. Water Research 10:377 (1976).

34. HOIGNE, J. & BADER, H. Effects of Carbonate on the Oxidizing Effect of Ozone and OH· Radicals. Vom Wasser 48:282 (1977).

35. CHEN, J.W. ET AL. Catalytic Ozonation in Aqueous Systems. AIChE 68th Annual Meeting, Los Angeles, CA (Nov. 1975).

36. SIERKA, R.A. The Effects of Sonic and Ultrasonic Waves on the Mass Transfer of Ozone and The Oxidation of Organics in Aqueous Solutions. Int'l Ozone Inst. 3rd Int'l Symp. & World Congress, Paris, France (May 1977).

37. PRENGLE, H.W. JR. ET AL. Oxidation of Refractory Materials by Ozone with Ultraviolet Radiation. Proc. 2nd Int'l Symp. on Ozone Techn., Montreal, Canada, Int'l Ozone Institute, Syracuse, NY (1976), p. 224.

38. LEE, M.K. ET AL. Reaction Kinetics of UV-Ozone with Organic Compounds in Hospital Wastewater. Int'l Ozone Institute Symp. on Advanced Ozone Techn., Toronto, Canada (Nov. 1977).

39. GLAZE, W.H. ET AL. A Comparison of Ozone and Ozone/UV for the Destruction of Refractory Organic Compounds in Water. Int'l Ozone Inst. Ozone Symp. & Expos., Los Angeles, CA (May 1978).

40. BUCK, B.J. ET AL. Ozonation Comes of Age. NY WPCA 50th Annual Meeting (Jan. 1978).

41. LAWRENCE, J. The Oxidation of Some Haloform Precursors with Ozone. Int'l Ozone Inst. 3rd Int'l Symp. & World Congress, Paris, France (May 1977).

42. JENKS, J.H. & HARRISON, B.L. Multi-Feature Reclamation Project Accomplishes Multi-Objectives. *Water & Wastes Engrg.* 14 (1977).

43. ROSS, W.R. ET AL. Studies on Disinfection and Chemical Oxidation with Ozone and Chlorine in Water Reclamation. Proc. 2nd Int'l Symp. on Ozone Techn., Montreal, Canada, Int'l Ozone Institute, Syracuse, NY (1976), p. 497.

44. LEAVITT, P. Water Reuse Through Ozone Sterilization. *Instrumentation in the Food and Beverage Industry*, ISA FID 726408 (1972).

45. VLAHAKIS, J.G. A Prototype Ozonation Waste Treatment System for the U.S. Army. Proc. 1st Int'l Symp. on Ozone for Water & Wastewater Treatment, Int'l Ozone Institute, Inc., Waterbury, CT (1975), p. 483.

46. MARSCHALL, K. Apparatus for Treating and Purifying Sewage, Particularly Sewage Contaminated with Detergents. U.S. Patent 3,822,786 (July 9, 1974).

47. HENDRICKSON, T.N. & DAIGNAULT, L.G. Treatment of Photographic Ferrocyanide-Type Bleach Solutions for Reuse and Disposal. *Jour. SMPTE* 82 (Sep. 1973).

48. DRAHUSHUK, R.L. & ROSEN, H.M. Resource Recovery with Ozonation Systems. Proc. 5th Annual Indus. Poll. Conf. & Expos., WWEMA, Atlanta, GA (April 1977).

49. LG Model Ozone Generators Brochure, #F-4266, Union Carbide Corporation.

50. ROSEN, H.M. Use of Ozone and Oxygen in Advanced Wastewater Treatment. *Jour. WPCF* 45:12:2521 (Dec. 1973).

51. LOWTHER, F.E. ET AL. Installation, Maintenance, and Reliability Considerations for Large-Scale Ozone Generators. Union Carbide Corporation White Paper (Mar. 1977).

52. BOLLYKY, L.J. Ozone Treatment of Cyanide and Plating Waste. Proc. 1st Int'l Symp. on Ozone for Water & Wastewater Treatment, Int'l Ozone Institute, Inc., Waterbury, CT (1975), p. 522.

53. STAHL, D.E. Ozone Contacting Systems. Proc. 1st Int'l Symp. on Ozone for Water & Wastewater Treatment, Int'l Ozone Institute, Inc., Waterbury, CT (1975), p. 40.

54. MASSCHELEIN, W. ET AL. Techniques for Dispersing and Dissolving Ozone in Water. Water & Sewage Works (Dec. 1975, Part 1; Jan. 1976, Part 2).

55. Ozonation Systems Brochure, #F-4117, Union Carbide Corporation.

56. ROSEN, H.M. Wastewater Ozonation: A Process Whose Time Has Come. Civil Engrg.-ASCE (Mar. 1976), p. 65.

57. Ozonation Systems - Accessory Equipment Brochure, #F-4265, Union Carbide Corporation.

58. JOHANSEN, R.P. & TERRY, D.W. Comparison of Air and Oxygen Recycle Ozonation Systems. Int'l Ozone Institute Symp. on Advanced Ozone Techn., Toronto, Canada (Nov. 1977).

59. Ozonation Systems - Oxygen Production and Supply Brochure, #F-4264, Union Carbide Corporation.

60. JOHANSEN, R.P. & TERRY, D.W. Comparison of Once-Through and Recycle Ozone Systems Using High Purity Oxygen. WPCF 50th Annual Conf., Philadelphia, PA (Oct. 1977).

TABLE 1

Applications of Ozone - Wastewater Renovation and Reuse

1. Disinfection
2. Direct Oxidation - Removal of BOD/COD/TOC
    a. Color and Turbidity
    b. Suspended Solids
3. Suspended Solids Removal by Flotation/Oxidation
4. Microflocculation
5. Specific Micropollutant Removal
6. Stripping for Volatile Organics Removal
7. Pretreatment Prior to Biological Treatment or GAC
8. Catalytic Ozonation

TABLE 2

Comparative Disinfection Efficiency: Ozone vs Chlorine

	Λ (mg/l)$^{-1}$ (min)$^{-1}$ at 5° C			
Agent	Enteric Bacteria	Viruses	Amoebic Cysts	Spores
O_3	500	5	0.5	2
Cl_2 as HOCl	20	1.0 Up	0.05	0.05
Cl_2 as OCl^-	0.2	<0.02	0.0005	<0.0005
Cl_2 as NH_2Cl	0.1	0.005	0.2	0.001

Λ = Specific susceptibility coefficient when organisms are compared.
Λ = Specific lethality coefficient when germicides are compared.

TABLE 3

Potable Water Disinfection
Pilot Plant, Laconia, NH - Lake Water

Organism	Seeding Level	Ozone Dose mg/l	Survivors	Comment
Coliforms	11×10^3/100 ml	1.19	<2	10 gpm Pilot Plant
Coliforms	Natural	0.2	<2	10 gpm Pilot Plant
Polio Virus Type II	2×10^3 pfu/ml	0.76	0	Batch
Coxsackie B-3 Virus	76×10^6 pfu/ml	1.13	0	10 gpm Pilot Plant

Contact Time All Data - 1.5 minutes

TABLE 4

Dallas Water Reclamation Study, Disinfection Data
Ozone Dose 5 mg/l, 25 gpm Pilot Plant

BACTERIA

Feed	A Filtered Activated Sludge	B Physical-Chemical (P-C) Alum, Densator	C (P-C)/Activated Carbon
No. of Tests	3	8	1
Avg. Influent Quality			
BOD, mg/l	<10	<10	<10
COD, mg/l	20	24	9
SS, mg/l	4	9	1
NH_3 - N mg/l	2.4	1.4	1.9
Organic - N mg/l	3.1	2.4	1.8
pH	7.3	7.9	7.4
Total Bacteria/100 ml	2.8×10^6	2.6×10^6	0.1×10^6
Total Coliform/100 ml	7.2×10^3	-	60×10^3
Disinfected Effluent			
Total Bacteria/100 ml*	37×10^2	80×10^2	0
Total Coliform/100 ml	5	-	0

BACTERIAL VIRUSES

Feed	A	B	C
No. of Tests		2	1
Avg. Effluent Quality			
BOD, mg/l		<10	<10
COD, mg/l		21	9
SS, mg/l		7	1
NH_3 - N mg/l		3.6	1.9
Organic - N mg/l		2.1	1.8
pH		7.4	7.4
f 2-phage - pfu/ml seeded		1700	52,500
Disinfected Effluent			
f 2-phage - pfu/ml seeded No. Samples/pfu/sample		13/0, 3/2	6/0

---

* Primarily gram positive rods

TABLE 5

Ozonation Results - Dallas Pilot Plant

Effluent Source	Parameter	Treated Effluent Quality	% Reduction
Alum Clarified Activated Sludge	Color (cu)	5	75
	Turbidity (JTU)	0.4	73
	COD (mg/l)	13	32
Filtered Activated Sludge	Color (cu)	-	69
	Turbidity (JTU)	-	20
Nitrified Activated Sludge	SS (mg/l)	1.5	80
	COD (mg/l)	19	17.4
	BOD_5 (mg/l)	5.8	31.8

TABLE 6

Ozone Process Effectiveness Summary
Chino Basin Pilot Plant

	Average Quality After Treatment		
Water Quality Parameter	Secondary Treatment	10 mg/l, 10 min. Ozonation	% Removal
Turbidity (TU)	5.5	2.8	49
Color (scu)	46	14	70
Suspended Solids (mg/l)	12	5	58
COD (mg/l)	87	44	49
Coliform Bacteria (MPN/100 ml)	380,000	1,900	99.5
Virus* (PFU/100 gal)	45,000	45	99.9

* Based on expected levels of enteric virus in CBMWD effluent and observed percent removals of seeded poliovirus.

TABLE 7

Effect of Ozonation on Chlorinated Pesticide Removal
Settled Trickling Filter Effluent

Pesticide	Absorbed Ozone, mg/l	Pesticide Concentration Pre-O_3	Pesticide Concentration Post-O_3	% Removal
γBHC	8.8	1.32	0.88	33
Dieldrin	18.3	1.30	0.66	49
DDT	11.7	2.00	0.54	73
TDE	11.7	2.00	0.62	69

TABLE 8

PAH Oxidation
30 Minutes Ozonation
0.4 ± .05 mg/l O_3, 3 μg/l PAH

	% Average Decrease in River Water	% Average Decrease in Drinking Water
Pyrene	84	100
1,2 Benzanthracene	84	100
3,4 Benzpyrene	100	100
3,4 Benzfluoranthene	43	61
11,12 Benzfluoranthene	28	62

TABLE 9

OEBAC - Ozone Enhanced Biologically Active Carbon
Ozone Oxidation and DO Enhancement Prior to GAC Adsorption

- Ozonation produces smaller, more biodegradable organics.
- A biological film established on the GAC converts BOD to CO_2 and H_2O.
- Carbon pores not blocked by the biological film - Results in increased GAC availability for adsorption of non-biodegradable organics.
- TOC and toxic organic chemical removal efficiency is high.
- No excessive backwash requirements or biomass sloughing problems.

TABLE 10

Advantages of OEBAC

- More effective removal of biodegradable and non-biodegradable organics than by GAC or ozone alone.
- Increased organic loading compared with GAC significantly.
- Extends time between necessary carbon regeneration (periods of 1 month have been extended to more than 3 years).
- Applicable to both water and wastewater treatment.
- Significantly reduces capital, operating and maintenance costs of GAC systems.
- Prevents anaerobic conditions and resulting problems.
- Ammonia removal (biological nitrification) possible.

TABLE 11

EFFECTIVENESS OF THE ROUEN PLANT IN REMOVING ORGANICS

Compound	Approximate Concentration, μg/l			
	Raw	After Pre-ozone	After Filt+AC	After Post-ozone
isoalcane >C 8	80	17	13	2
methylcyclohexane	14	10	5	5
isoalcane >C 8	16	8	6	t
isoalcane >C 8	72	40	17	5
isoalcane >C 8	4	4	3	t
n-nonane	4	t	t	t
isoalcane >C 8	4	t	t	t
n-decane	2	t	t	t
isoalcane >C 10	2	t	t	t
isoalcane >C 10	0.4	t	t	t
n-undecane	1	0.5	0.5	t
	18	2.5	1	1
isomer of 4-tert-	22	2.5	1	0.5
butylcresol or of	6.5	1	0.5	0.4
tert-butylanisole	6	1	0.5	0.5
	0.8	0.8	0.8	0.8
bicyclohexyl	70	33	30	30
not identified	10	t	t	t
n-tetradecane	0.7	0.3	0.3	0.3
not identified	0.4	0.4	0.4	0.4
n-pentadecane	0.5	0.2	0.2	0.2
isoalcane > C 15	0.2	t	t	t
n-hexadecane	1.5	0.6	0.6	0.6
n-heptadecane	1	0.3	0.3	0.3
not identified	0.4	t	t	t
n-octadecane	1.5	0.5	0.5	0.5
DIBP**	7	3.5	3	3
DIBP**	4	2.5	2	2
DBP*	650	390	320	240
n-eicosane	1.2	t	t	0
isoalcane ≥C 20	0.4	0.4	t	0
isoalcane ≥C 20	0.4	t	t	0
isoalcane ≥C 20	0.4	t	t	0
not identified	3	2	1	0.1
not identified	t	t	t	t
isoalcane ≥C 20	0	t	t	0.2
DEHP***	330	220	130	117
Σ (less DBP + DEHP)....	355.3	131.0	86.6	52.8
% Removal by process		63%	34%	39%
by train		63%	76%	85%

* DBP = Dibutyl phthalate
** DIBP = Diisobutyl phthalate
*** DEHP = Di-2-ethylhexyl phthalate
t = trace

Note: This is not an all-inclusive list of all organics present in the Rouen water, but rather only those organics detectable by gas chromatography. DBP and DEHP are included, since they were added to serve as chromatographic internal standards.

TABLE 12

RFI* Values - Selected Compounds [37]

Compound	Formula	RFI (O_3)	RFI (O_3 + UV)
Ethanol	CH_3CH_2OH	245	47
Acetic Acid	CH_3CO_2H	>1000	41
Ferricyanide	$Fe(CN)_6^{-3}$	270	8.6
Glycerol	$CHOH(CH_2OH)_2$	112	7.4
Palmitic Acid	$CH_3(CH_2)_{14}CO_2H$	27.3	7.2
Glycine	$H_2NCH_2CO_2H$	19.7	6.0

* RFI = Refractory Index. A measure of relative resistance to ozone oxidation based on half-life conversion to CO_2 and H_2O.

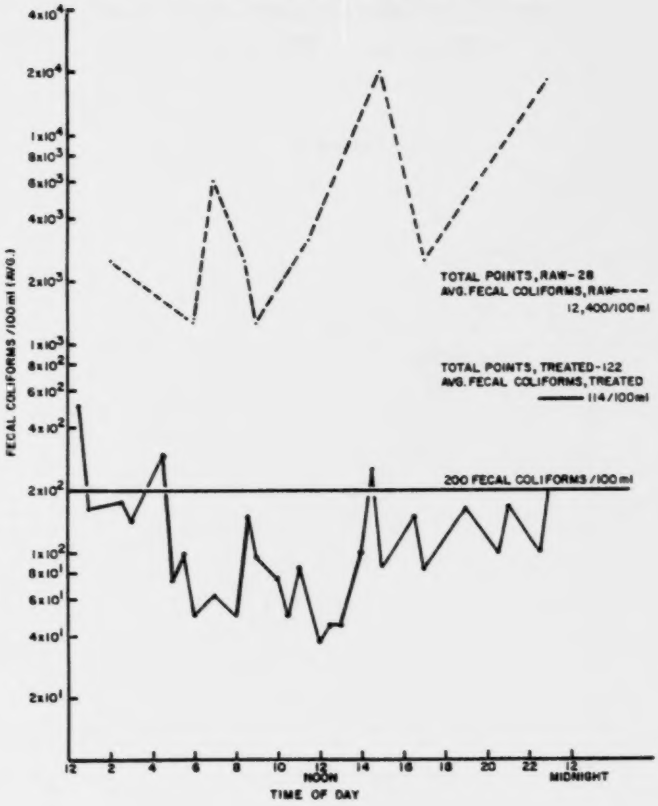
TABLE 13

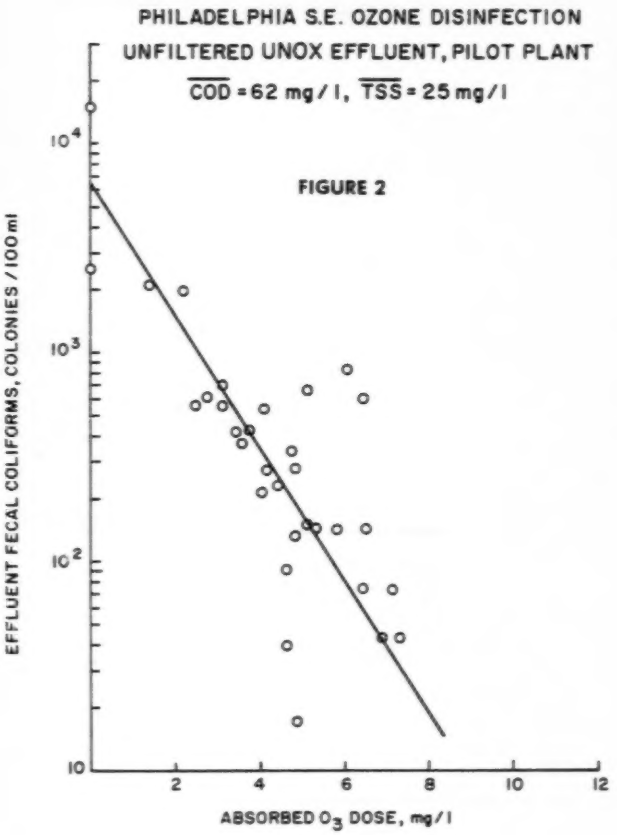
Examples of Reuse Systems Employing Ozone

Direct/Indirect Potable	Reference	Comments
Denver, CO	-	• 1 mgd pilot plant planned for 1980 startup.
Washington, DC	-	• 1 mgd pilot plant under construction to treat Potomac River and/or Blue Plains wastewater for reuse.
Orange County, CA	42	• Water Factory 21 - OEBAC operational at 4 mgd.
Chino Basin, CA	16	• 5 mgd plant in startup.
Pretoria, S.A.	43	• National Institute of Water Research studies for Windhoek and Pretoria, disinfection and pesticide oxidation.
Technion, Israel	30	• Tertiary polishing, OEBAC pilot program.
Industrial/Agricultural		
Gerber Foods	44	• Recycle of baby food bottle wash water. Operational over 10 years.
U.S. Army	45	• Reuse of mobile hospital wastewater for non-potable purposes. UV/Ozone pilot studies.
Car Wash	46	• Reuse of washwater practiced as part of a system used in Austria.
Photographic	47	• A number of systems operable for treatment of wastes and reuse of chemicals.
Arab Emirates	-	• Several systems in design for disinfection of wastewater for irrigation purposes.

FIGURE 1

OZONE DOSE 5mg/l-OZONATION OF OXYGEN ACTIVATED SLUDGE EFFLUENT FROM SECONDARY CLARIFIER





PHILADELPHIA S.E. OZONE DISINFECTION
UNFILTERED UNOX EFFLUENT, PILOT PLANT
$\overline{COD}$ = 62 mg/l, $\overline{TSS}$ = 25 mg/l

FIGURE 2

PHILADELPHIA S.E. OZONE DISINFECTION
FILTERED UNOX EFFLUENT, PILOT PLANT
COD = 58 mg/l, TSS = 5 mg/l

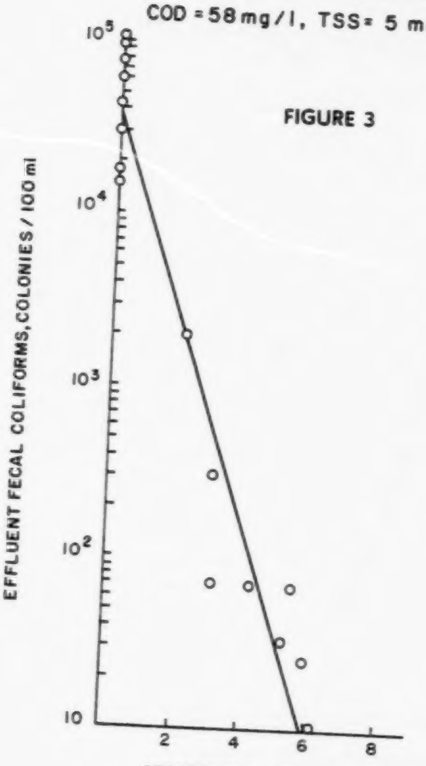
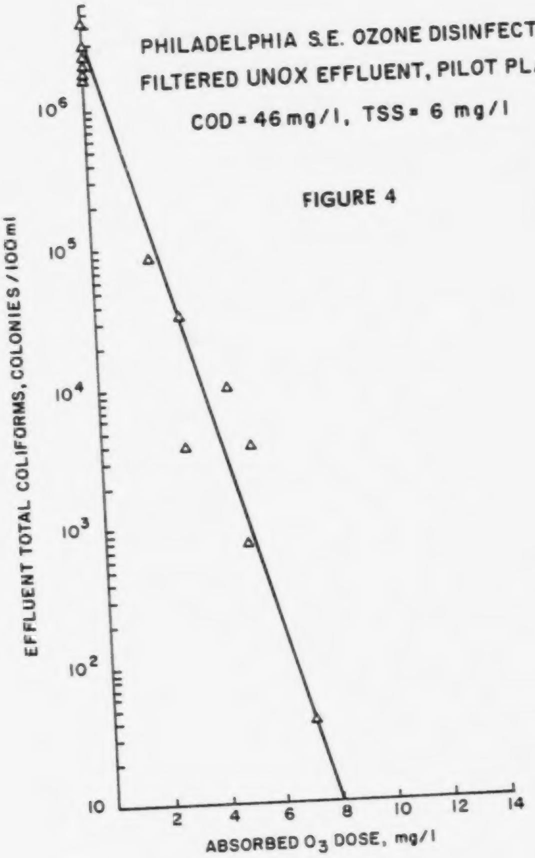


FIGURE 3



PHILADELPHIA S.E. OZONE DISINFECTION
FILTERED UNOX EFFLUENT, PILOT PLANT
COD = 46 mg/l, TSS = 6 mg/l

FIGURE 4

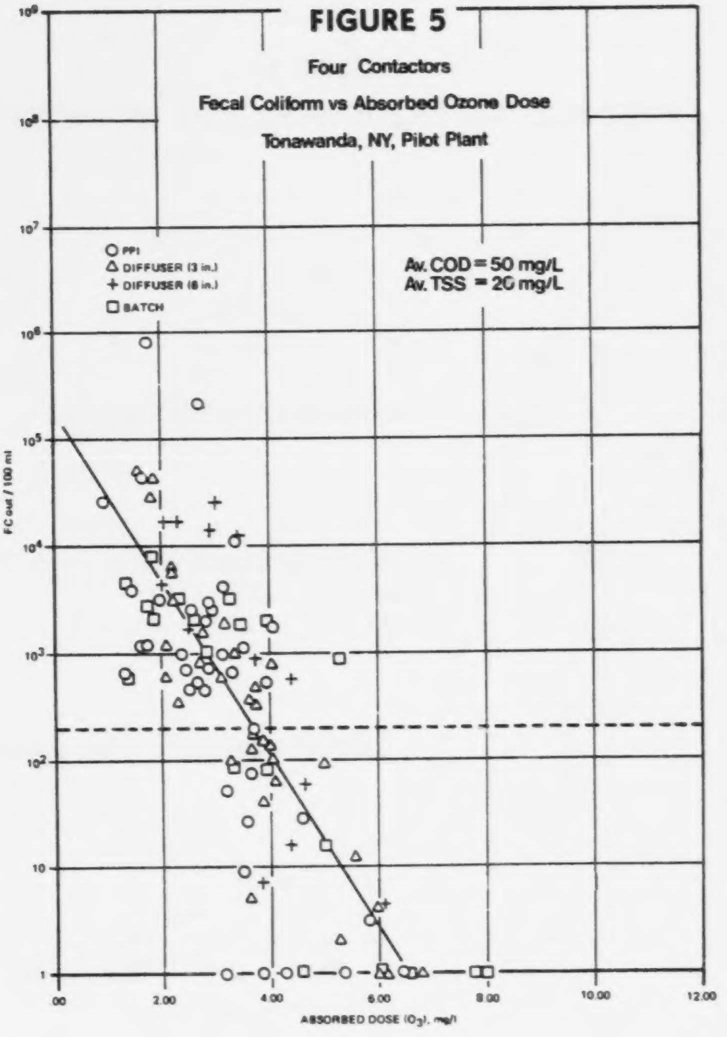


FIGURE 5

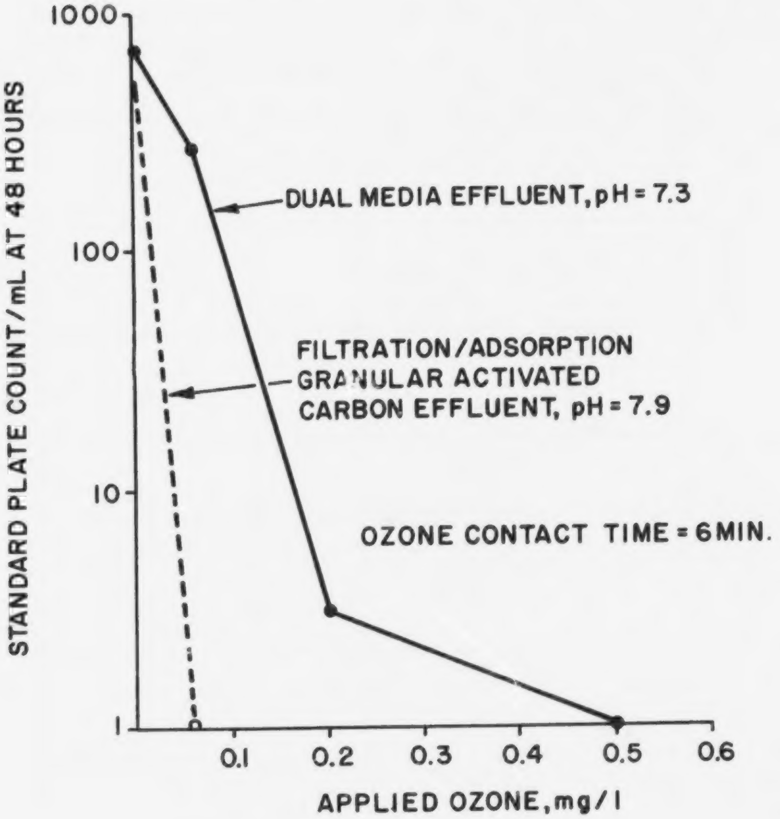
Four Contactors

Fecal Coliform vs Absorbed Ozone Dose

Tonawanda, NY, Pilot Plant

FIGURE 6

POTABLE WATER DISINFECTION WITH OZONE



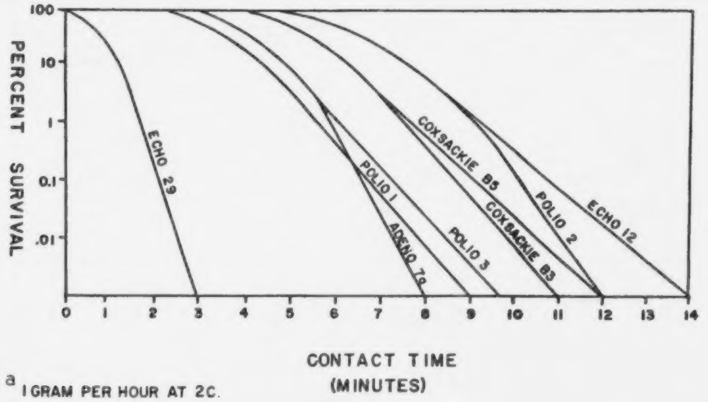


FIGURE 7

Inactivation of Viruses by Ozonation
in Saugatucket River Water[a]

[a] 1 GRAM PER HOUR AT 2C.

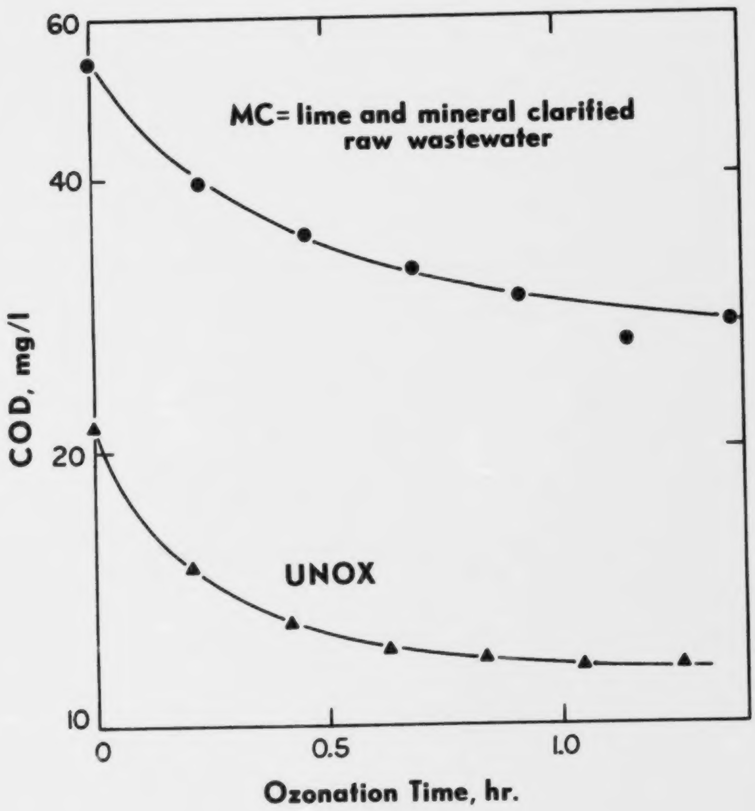


FIGURE 8

Typical COD Reduction Curves for MC and UNOX Pretreated Feeds: Log COD vs Reaction Time

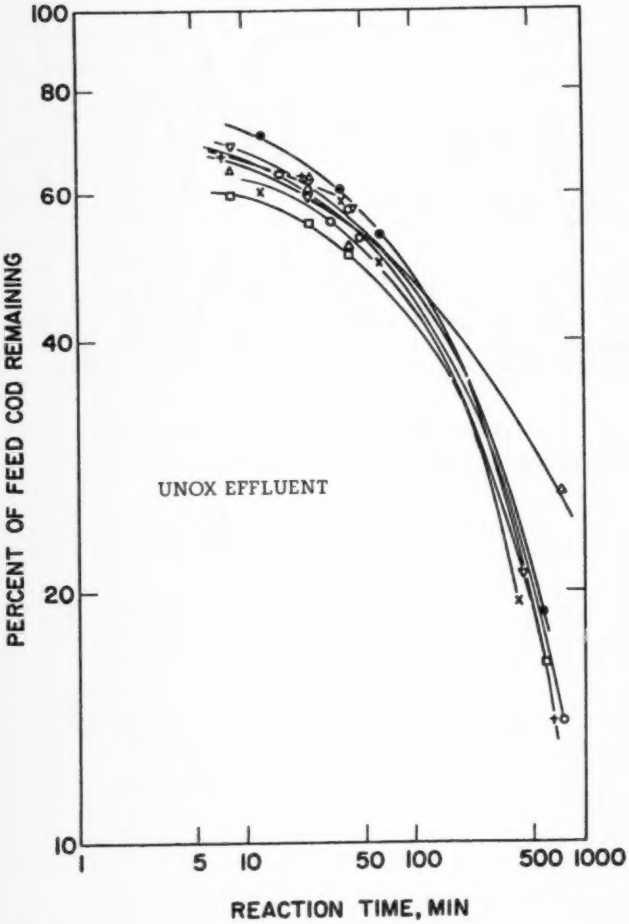
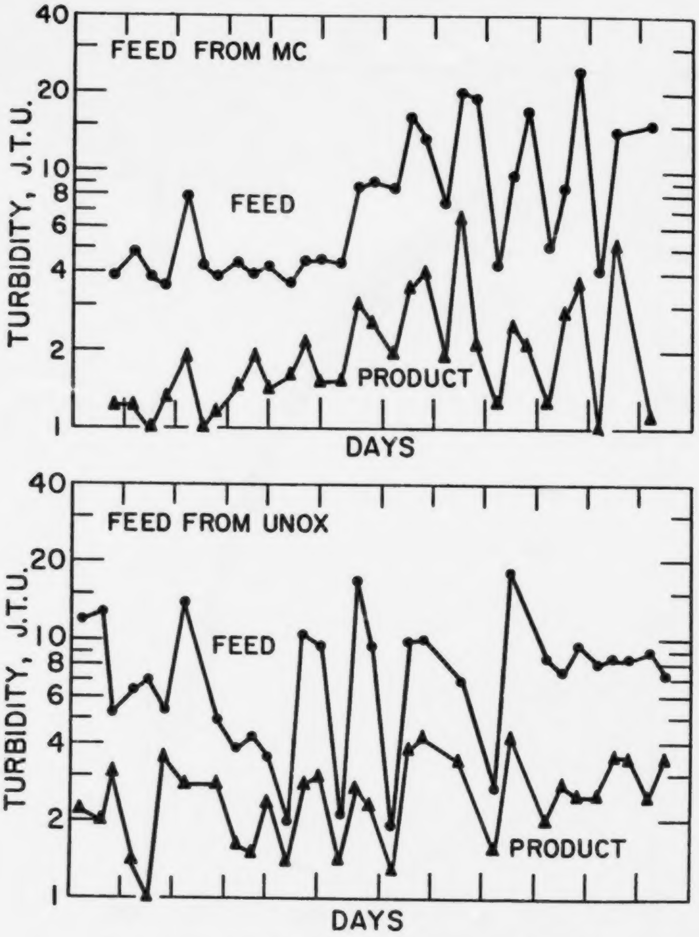


FIGURE 9

Extended COD Reduction Curves for UNOX Pretreated Feeds



(MC = Lime and mineral clarified raw wastewater)

FIGURE 10

Turbidity Reduction for MC and UNOX Pretreated Feeds

FIGURE 11

OZONE DOSE 5mg/l-OZONATION OF OXYGEN ACTIVATED SLUDGE EFFLUENT FROM SECONDARY CLARIFIER

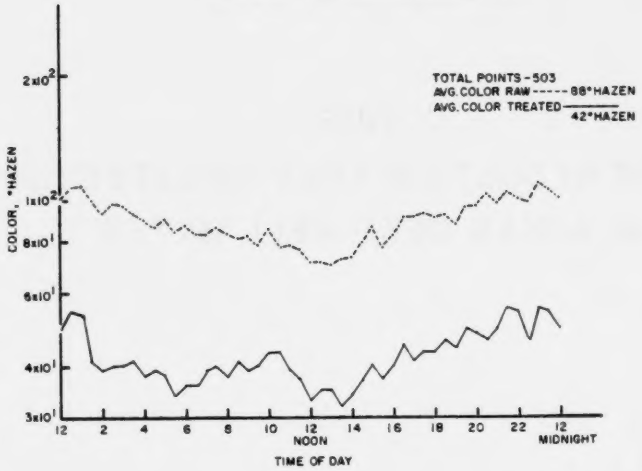


FIGURE 12

OZONE DOSE 5mg/l-OZONATION OF OXYGEN ACTIVATED SLUDGE EFFLUENT FROM SECONDARY CLARIFIER

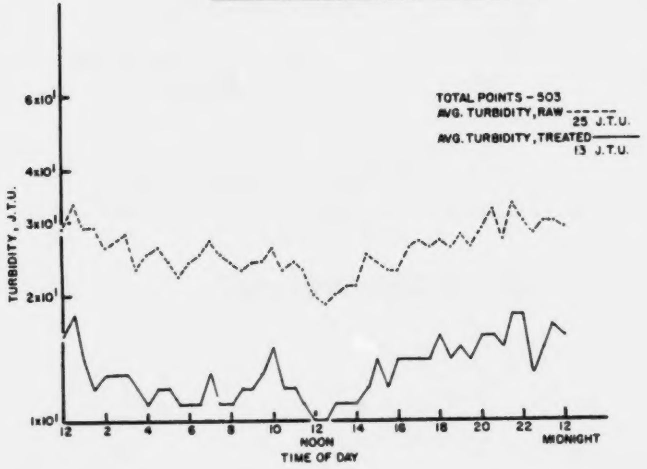


FIGURE 13

COLOR REDUCTION TREATABILITY STUDY ALASKAN DEEP WELL WATER

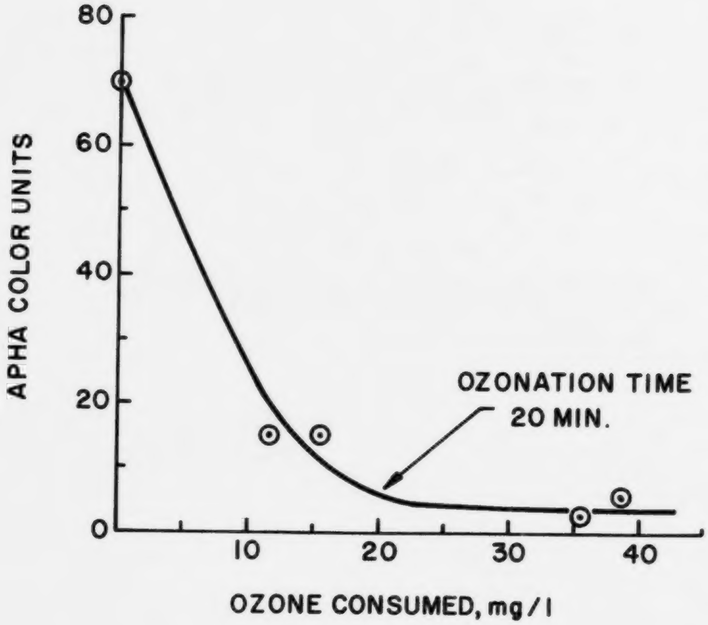
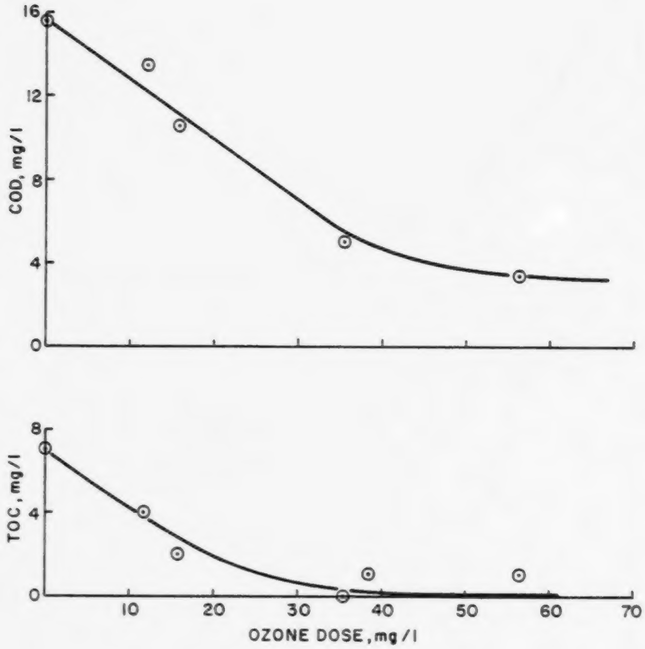


FIGURE 14

ORGANIC REDUCTION TREATABILITY STUDY ALASKAN DEEP WELL WATER



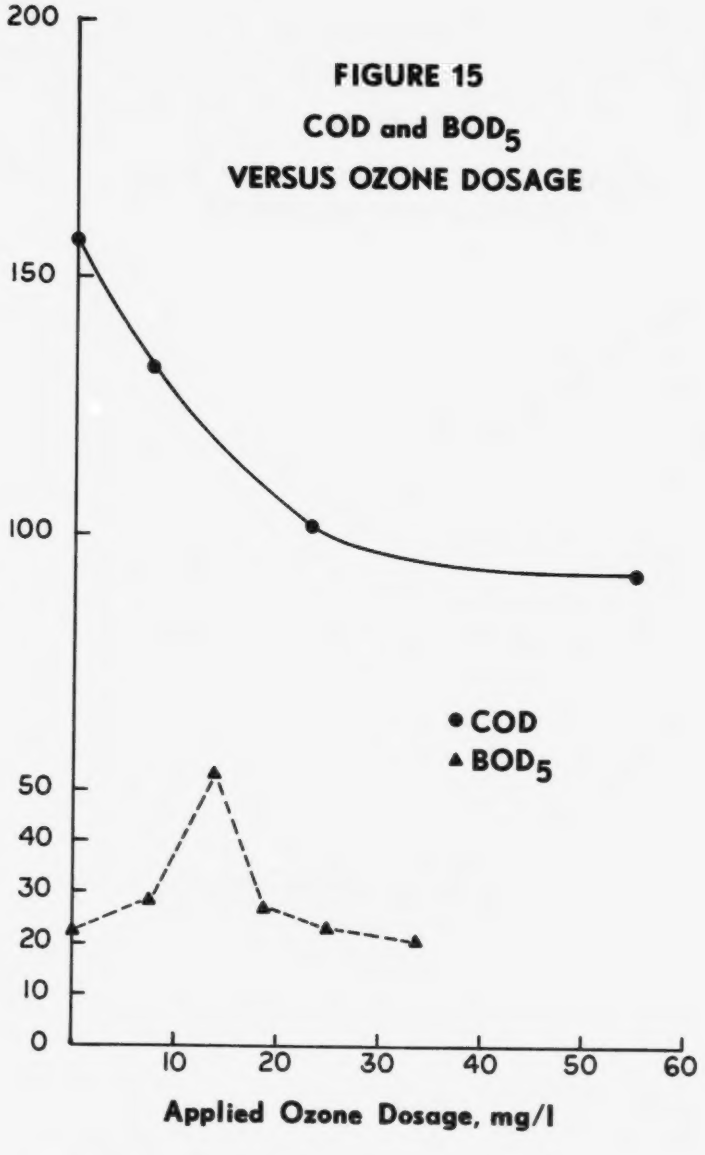


FIGURE 15
COD and BOD_5
VERSUS OZONE DOSAGE

FIGURE 16

BAC

ACTIVATED CARBON LOADING

LENGG PLANT, ZURICH, SWITZERLAND

DMFE = DIMETHYLFORMAMIDE EXTRACT

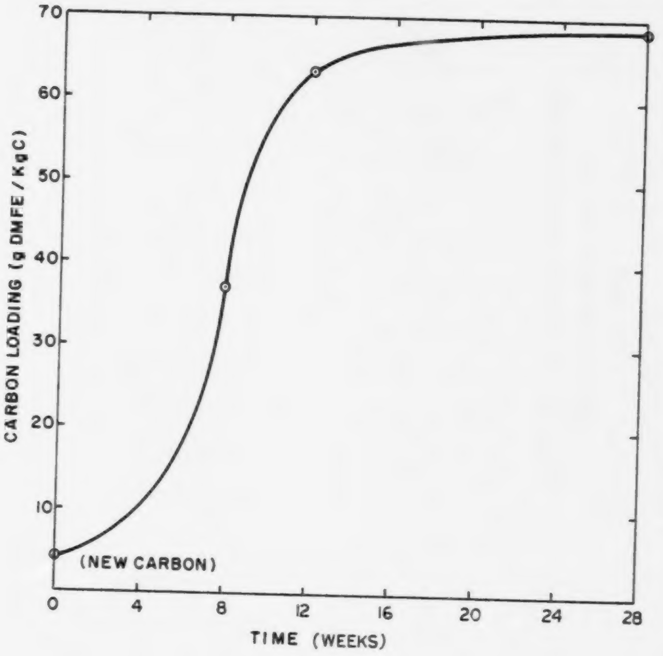


FIGURE 17
BAC
COD REMOVAL EFFICIENCY
AND
U.V. ABSORPTION (254 nm)
LENGG PLANT, ZURICH, SWITZERLAND

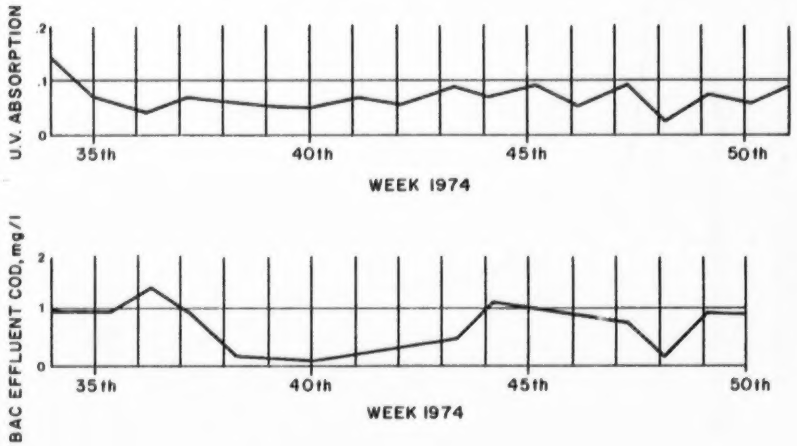
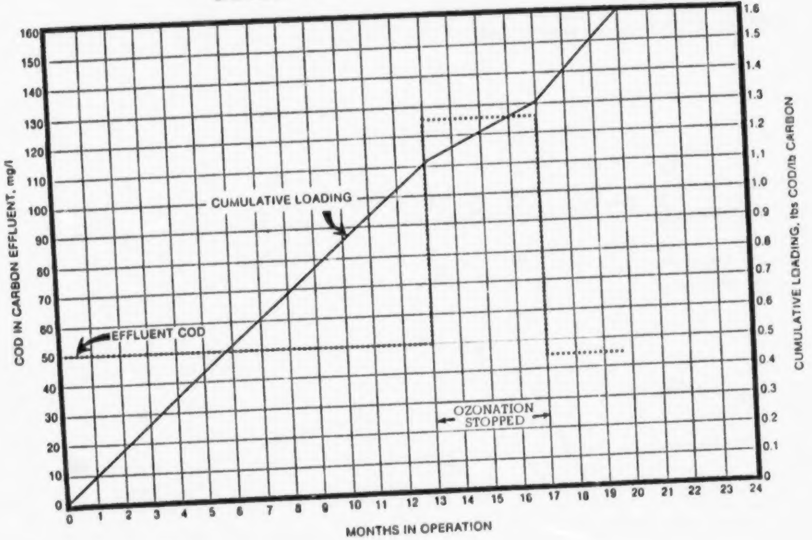


FIGURE 18

BAC Performance At Cleveland Regional Sewer District



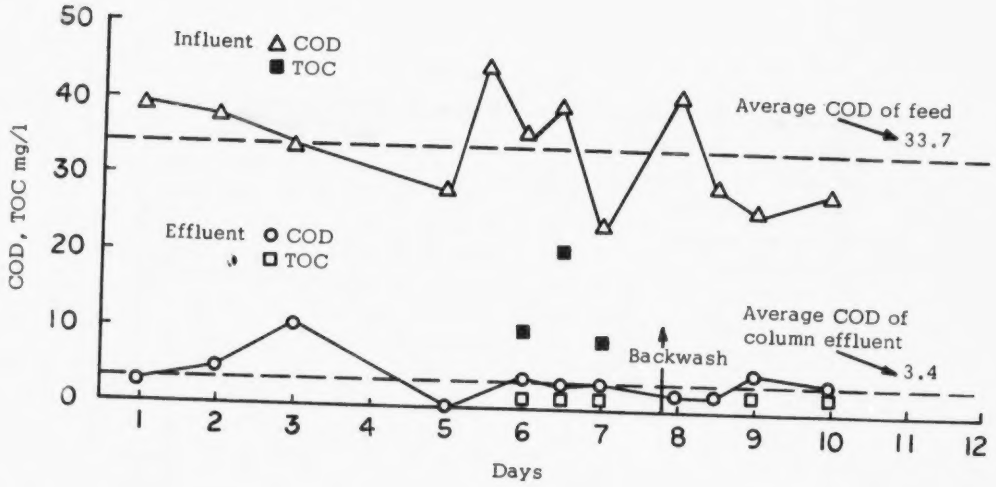


FIGURE 19

Biologically Extended Activated Carbon Treatment of Ozonated Effluent

FIGURE 20

Synthetic RO Permeates of Hospital Wastewater
TOC_t/TOC_o Ratio as a Function of Reaction Time

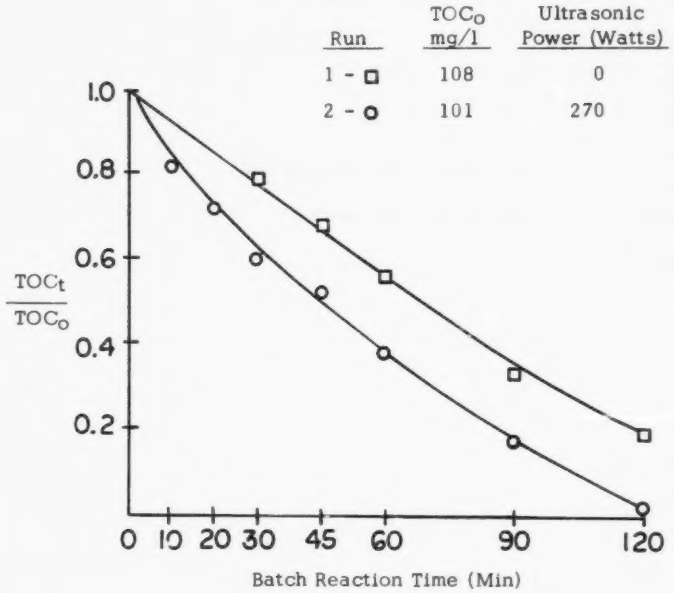
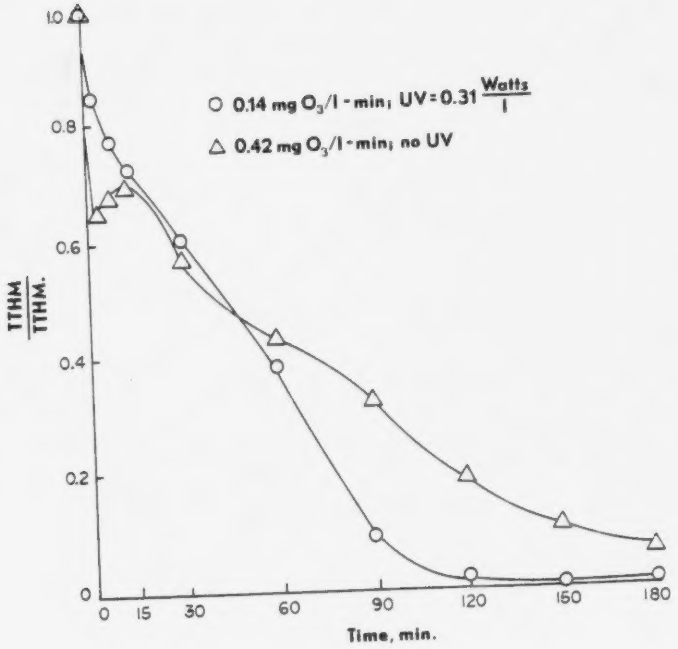


FIGURE 21

Normalized THM Formation Potential: Ozone Destruction of THM Precursors - Caddo Lake Water



COMPOSITE MEMBRANE FOR WASTE WATER RECLAMATION

Richard G. Sudak
President

Ronald L. Fox
Research Chemist

Membrane Systems, Inc.

The generally accepted date for the beginning of commercial application of reverse osmosis membranes is 1970. The first market of any magnitude was the industrial pretreatment of process water in the electronics industry. The use of reverse osmosis for the production of potable water from brackish wells followed, but this has been limited to locations where the lack of a dependable potable water supply retarded development of valuable real estate. The process is now being introduced on a commercial scale as an alternative to distillation for the demineralization of seawater, and initial results would indicate that reverse osmosis should be regarded as a serious competitor. However, the severity of a seawater environment and the consequent economics will limit the use of either process for the foreseeable future. Another obvious source of potable and industrial waters is the renovation of waste waters by reverse osmosis. This assumes that the valid issues of health considerations may be successfully resolved It would appear that some of these problems may have been overcome already in that large scale plants are in operation or in planning; e.g., Water Factory 21 and the Yuma Desalting Plant.

Until recently, the reverse osmosis process has relied solely on the use of asymmetric membranes which were prepared from either cellulose acetate or polyamide. The cellulose acetate membranes were packaged in the spiral wound, tubular and hollow fine fiber configuration; and the polyamide was packaged in the hollow fine fiber configuration. Table 1 shows some of the operating limits of these asymmetric membranes. It is evident that the number of feedwaters that can be desalinated with these membranes is limited and in many cases extensive pretreatment is required. Unfortunately, waste waters emanate from a wide variety of sources, and it would appear that in order to be more effective in the treatment of waste waters, membranes with more versatility will have to be developed. This could be done with a single membrane having a wider range of operating limits or, as is more likely, with a family of membranes to meet the variable needs.

Asymmetric membranes are fabricated from various homopolymers. The formulation and casting conditions largely determine their reverse osmosis properties. In the last decade it was determined that an asymmetric membrane consists of a relatively thin, dense skin supported by an amorphous underlayer. Figure 1 shows this for a cellulose acetate sheet membrane. In general, most sheet membranes are cast on a fabric carrier material which is a mechanical aid to assist in handling. Once the nature of asymmetric membranes was understood, the logical extension was the concept of composite membranes. Figure 2 shows a porous support of one polymer cast on a fabric carrier material and a membrane barrier layer of another polymer formed on the porous support.

The first composite membrane was developed at the North Star Research Institute and designated as the NS-1 membrane (1). Polysulfone was used as the porous support and the membrane barrier layer was formed by the interfacial polycondensation of polyethylenimine (PEI) and toluene-2,4-diisocyanate (TDI). The resultant polyurea had all the advantages and disadvantages of the hollow fine fiber polyamides. Another composite developed at North Star was the NS-200 membrane (2). This membrane barrier layer is formed on a polysulfone porous support by the in-situ condensation of furfuryl alcohol into furan. Additional composite membrane work was done at the Fluid Systems Division of UOP where the PA-100 membrane (3) was formed on polysulfone by the interfacial polycondensation (4) of PEI with isophthaloyl chloride (IPC) to form a polyamide membrane barrier layer (5). The substitution of IPC for TDI gave an improved flux while retaining good rejection characteristics. All of the above membranes have one thing in common in that they are sensitive to residual chlorine in the feedwater.

In the case of membranes formed from PEI, it has been theorized that the residual chlorine sensitivity is the result of secondary amines in the polymer backbone. In an attempt to improve the resistance to residual chlorine, Fluid Systems developed the PA-300 membrane (6). This membrane also utilizes a polysulfone porous support and the membrane barrier layer is formed from the interfacial polycondensation of epiamine and IPC. Epiamine is a trade name for a water soluble polymer formed by the condensation reaction of polyepichlorohydrin and ethylene diamine. It differs from PEI in that secondary diamines are not in the polymer backbone but are pendant groups. This membrane proved to be more resistant to residual chlorine in that considerably more time is required to achieve the same level of performance degradation that would be achieved by PA-100 membrane at the same concentration of residual chlorine.

Another membrane which has been designated as RC-100 by Fluid Systems is formed on polysulfone by the interfacial polycondensation of epiamine and TDI. A summary of the polymer systems discussed above is shown in Table 2. It is understood that the NS-200 membrane has been evaluated for possible commercial production and has been found to be unstable under long term operating conditions even with dechlorinated feedwaters. Of the membranes mentioned, only the PA-300 and the RC-100 type of membranes are commercially available at this writing. In any event, only a small number of polymer systems have been evaluated for composite membrane application and it is suggested that this represents a small fraction of the possibilities available to polymer chemists.

The PA-100 system has been investigated at Membrane Systems and has been found to be extremely versatile. By varying the concentration of reactants it can be used from low pressure brackish water applications to the desalination of seawater. One formulation will give a membrane that will produce 20 gallons per square foot per day (GFD) at 97% salt rejection when operated at 250 psig on a feedwater containing 5,000 mg/l of sodium chloride at a temperature of 75°F. Another variation will give a membrane that will produce 20 GFD at 99.2% rejection when operated at 800 psig on a feedwater containing 35,000 mg/l of sodium chloride at a temperature of 75°F. The PA-300 membrane shows similar versatility with reactant concentration variation. At the low pressure test conditions specified above, fluxes of 16 GFD and 98% rejection were obtained. Under seawater conditions fluxes of 20 GFD and 99% rejection have been obtained.

The PA-100 and PA-300 membranes have the following advantages over cellulose acetate membranes:

1. They can operate over a broader range, i.e., pH 3-12. However, it is noted that flux and rejection vary as a function of pH and the optimum rejection is found between pH 4-6.

2. Flux decline, or compaction, is of less consequence. A log-log compaction slope of -0.01 at 1,000 psig and 25°C have been shown, whereas recently developed cellulose acetate membranes show a compaction rate of -0.04.

3. There is no evidence of biological degradation in the limited data available.

4. The membrane has been shown to be stable to alcohols, esters, ketones and hydrocarbons and demonstrates better rejections of these compounds.

5. They can be operated at elevated temperatures, i.e., 140°F compared to 95°F for cellulose acetate.

As previously mentioned, both of these membranes are susceptible to attack by residual chlorine.

To eliminate the susceptibility of polyamides to residual chlorine, several membranes have been prepared with the difunctional secondary amine, piperazine. By reacting piperazine with IPC, the secondary amine is converted to a tertiary amine and, theoretically, the resultant polyamide is resistant to residual chlorine. At Membrane Systems, this technique was used to form the membrane barrier layer on polysulfone. Short term testing showed that this composite membrane was indeed resistant to residual chlorine. However, the best flux and rejection obtained at low pressure brackish water test conditions were 5 GFD and 90% rejection.

As noted above, the prepolymer, epiamine, has been used successfully in the PA-300 membranes. It was theorized that piperazine, instead of ethylene diamine, could be reacted with polyepichlorohydrin to form a poly(epichlorohydrin-secondary diamine) as the prepolymer. This would then be reacted with IPC to form a polyamide membrane barrier layer without a secondary amine in either the backbone or the crosslinking sites. The prepolymer has been synthesized at Membrane Systems and composite membranes have been prepared which, under low pressure brackish water test conditions, give a flux of 17 GFD and a rejection of 97%. While the membrane does show an improved resistance to residual chlorine, it is still susceptible to attack. Work on this system is in progress as of this writing and conclusions are not available.

Another approach utilizing piperazine has been attempted at the North Star Division of the Midwest Research Institute. Prepolymers were formed by reacting either trimesoyl chloride, cyanuric chloride or phosphorous oxychloride with piperazine (7). The product was then reacted with IPC to form the membrane barrier layer. Indications are that this membrane is resistant to residual chlorine and reports of preliminary work show interesting properties for seawater desalination. When tested at 1,000 psig on synthetic 3.5% seawater, typical results were 13.9 GFD and 99.2% rejection. It is suspected that these membranes would also be capable of operation at extended pH ranges and higher temperatures while not being susceptible to biological attack.

To date, the majority of composite membranes have been formed on polysulfone porous support. While polysulfone has served its purpose, it has inherent limitations which prevent its use in the development of a broad range of composite membranes. Polysulfone has excellent thermal stability and it can be used under continuous service at temperatures up to 170°C. It is highly resistant to oxidizing agents, mineral acids, alkali and salt solutions. Resistance to aliphatic hydrocarbons and detergents is good but it is attacked by chlorinated hydrocarbons and aromatic hydrocarbons as well as polar organic solvents such as esters and ketones. It has fair to good resistance to most alcohols and ethers.

Since the techniques for casting polysulfone are well known, it is not surprising that it was the first choice for use as a porous support in composite membranes. The low temperature used in the formation of the NS-1 membrane barrier layer and the intermediate temperature used in the formation of NS-200 membrane barrier layer are compatible with the nature of polysulfone. In addition, the ultimate use for the membrane (desalination of brackish and seawaters) was also compatible with the chemical and mechanical properties of the support.

Development of other membranes has been impeded due to the lack of a wide range of porous supports. For example, the in-situ formation of a polyimide membrane barrier layer requires a higher temperature capability than that possessed by polysulfone. Another example is the thin film deposition of a homopolymer from a chlorinated hydrocarbon solvent.

Recent work at Membrane Systems has resulted in the preliminary development of the following porous support materials:

1. Polycarbonate
2. Polyphenylsulfone
3. Polyimide
4. Polyvinylidenefluoride
5. Polyphenylquinoxaline

Some of the interesting physical and chemical properties of these polymers are shown in Table 3 (8). To date, it has been demonstrated that a porous support can be cast from each of these polymers and that the porous support can be successfully incorporated into a PA-100 type composite membrane. As can be seen from Table 3, the above porous supports do extend the range of possibilities that are now open to membrane scientists with imagination.

To this point, the discussion has been limited to the role of composite membranes in desalination with reverse osmosis. In order to produce an effective composite membrane, it is necessary to have a porous support with a small mean pore diameter and a large porosity (number of pores). This necessity is dictated by the fact that the support must not only handle permeate flow with relatively little resistance, but it must truly support the thin film membrane barrier layer without rupture. Both the mean pore diameter and the porosity can be altered by variation of casting solution formulation and/or casting conditions. By tailoring the pore diameter, it is possible to use any of the six porous supports mentioned above as an ultrafiltration membrane. If desalination of a waste water is not required and the only requirement to reclaim a waste water is the removal of suspended solids, then this can be done more economically with ultrafiltration membranes.

In conclusion, the following observations about composite membranes should be made:

1. The concept of thin film composite membranes expands the number of possibilities available to custom fabrication of membranes for specific applications in waste water reclamation.
2. While initial developments in composite membranes are very promising, it is evident that only a very small number of possibilities has been explored.
3. As the needs in the field of waste water reclamation are clarified, efforts of membrane scientists will

focus on the technical and economic feasibility of adapting composite membrane technology to the problem.

ACKNOWLEDGMENT

The authors wish to express their appreciation to the U.S. Department of Interior, Office of Water Research and Technology for helpful suggestions, guidance and financial support in their work on composite membranes.

REFERENCES

1. Cadotte, J.E., et.al., In-situ Formed Condensation Polymers for Reverse Osmosis Membrane, Quarterly Report OSW Contract No. 14-30-2883, North Star Research, (April 1972)

2. Cadotte, J.E. et.al., In-situ Formed Condensation Polymers for Reverse Osmosis Membrane, Final Report OWRT Contract 14-30-2883, North Star Research, (Feb. 1975)

3. Riley, R.L., et.al., Research and Development on a Spiral Wound Membrane System for Single Stage Seawater Desalination. Final Report OWRT Contract No. 14-30-3303, Fluid Systems Div, UOP, Inc.,(1974)

4. Enkelman, V., et.al., Mechanism for Interfacial Polycondensation and the direct synthesis of Stable Polyamide Membrane, Macromol.Chem. 177,3177 (1976)

5. Wittbecker, E.L. & Morgan, P.W., Interfacial Polycondensation, I-X, J. Polymer Sci., XL, 289-418 (1959)

6. Riley, R.L., et.al., Spiral Wound Poly(Ether-Amide) Thin Film Composite Membrane Systems, First Desalination Congress of the American Continent, Mexico City, Fluid Systems Div., UOP, Inc. (1976)

7. J.E. Cadotte, et.al., Research on In-situ Formed Condensation Polymer for Reverse Osmosis Membranes. Final Report OWRT Contract No. 14-34-0001-6521, North Star Research Div., Midwest Research Inst. (1978)

8. Harper, C.A., Handbook of Plastics and Elastomers, McGraw-Hill Book Co., New York (1975)

TABLES

TABLE 1

ASYMMETRIC MEMBRANE OPERATING LIMITATIONS

	Cellulose Acetate	Polyamide
Temperature, °F	32-95	32-95
pH Range, Continuous	4-6	4-11
Residual Chlorine Sensitivity	NO	YES
Biological Attack	YES	NO

TABLE 2

Membrane	Reactants	Membrane Barrier Layer Polymer
NS-1	Polyethylenimine-Tolylene-2,4-diisocyanate	Polyurea
NS-200	Furfuryl Alcohol	Furan
PA-100 NS-100	Polyethylenimine-Isophthaloyl Chloride	Polyamide
PA-300	Epiamine-Isophthaloyl Chloride	Polyether/amide
RC-100	Epiamine-Tolylene-2,4-diisocyanate	Polyether/urea

TABLE 3

POROUS SUPPORTS

PHYSICAL AND CHEMICAL PROPERTIES

Material	Continuous Heat, °C	Acids	Bases	Solvents Aromatic	Aliphatic	Chlorinated	Ketones & Esters
Polysulfone	170°	G-F	G	X	G-F	X	F-X
Polycarbonate	125°	G-F	X	X	G	X	X
Polyphenylsulfone	260°	E	E	F	E	G-F	F-G
Polyvinylidenefluoride	170°	E	E	E	E	E	X
Polyimide	260°	G-X	X	E	E	E	E
Polyphenylquinoxaline	450°	G	E	E	E	X	E

E - Excellent, no discernible attack

G - Good, no significant attack

F - Fair, limited attack

X - Unacceptable

ILLUSTRATIONS

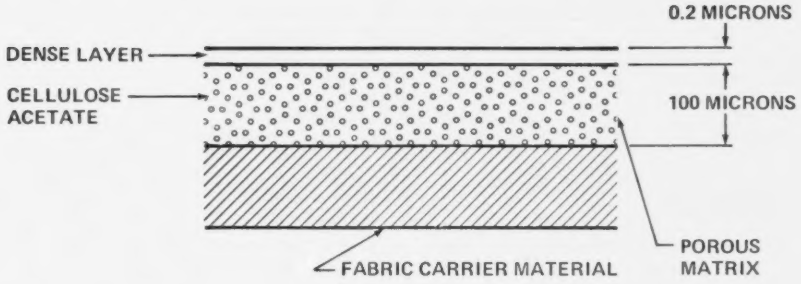


Figure 1. Asymmetric Membrane

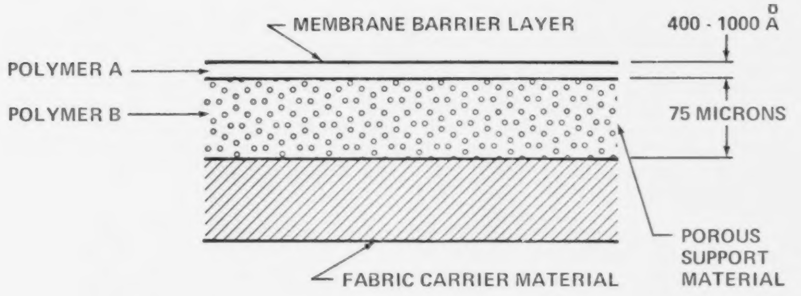


Figure 2. Composite Membrane

OZONE-ULTRASOUND TREATMENT OF A HOSPITAL WASTEWATER FOR REUSE

Raymond A. Sierka
Professor
Department of Civil Engineering
The University of Arizona

Robert L. Skaggs
Professor
Department of Engineering
The University of Nevada at Las Vegas

INTRODUCTION

The US Army provides field hospital medical facilities derived from a modular building block concept termed MUST; Medical Unit, Self-Contained, Transportable. An integral part of this system is the Water Processing Element (WPE) which will treat non-sanitary wastewaters emanating from the following MUST activity centers; showers, operating rooms, kitchen, x-ray laboratory and clinical laboratory. The current WPE process configuration includes equalization tankage, 40 mesh screening, ultrafiltration (UF), reverse osmosis (RO), ultraviolet (UV)/ozonation and hypochlorination.

The ultimate objective is to produce recycleable water, meeting US Drinking Water Standards, for use within the MUST complex. Additionally, 5 mg/l total organic carbon (TOC) and 10 mg/l chemical oxygen demand (COD) limits are imposed. A near term objective is to produce a treated composite wastewater acceptable for discharge to the environment. To meet these requirements, UF permeates after equalization and screening were to be ozonated in the presence of UV light, then passed through the hypochlorination operation prior to discharge.

The UV/ozonation process was relied upon to meet the organic quality standards outlined above regardless of the final disposition of the treated water. Since this portion of the system was not optimized, research was initiated to acquire design information on the effect that mechanically generated sonic and ultrasonic waves had; (1) in an ozone reactor during the oxidation of synthetic MUST Hospital Composite UF and RO permeates using low volumetric gas flow rates (i.e., volume of gas per minute per volume of liquid (VVM) of 0.1, 0.2, and 0.4 VVM, (2) on the mass transfer of oxygen into solution at the above VVM's, and (3) on the air-stripping of UF permeates at 45°C.)

EXPERIMENTAL

The wastewaters employed throughout the course of this research were synthetic UF and RO permeates manufactured from the ingredients listed in Table 1. These formulations were derived by the US Army Medical Bioengineering Research and Development Laboratory (USAMBRDL) from gas chromatograph mass spectrophotometer (GCMS) data on pilot plant UF and RO processed MUST wastewaters. For each day's operation, analytical reagent-grade chemicals were mixed in 64.0-l batches with City of Tucson, Arizona tap water serving as the solvent. The batch pH was adjusted to 9.0±0.1 with sodium hydroxide.

Figure 1 is a process flow diagram of the experimental apparatus employed. The reactor, a 6.0 in ID by 7.5 ft high section of glass pipe, had bottom and top closure plates constructed of 304 type stainless steel. The bottom plate served as acceptor for the sound wave generator*. Two lines (0.25 in ID) served as drain and sample ports for the reactor which was run in the semi-batch (i.e., continuous gas addition and fixed liquid volume) mode. Ozone and its carrier gas were admitted through one port of the top plate through tygon tubing to the gas sparger while the unreacted gases, after sparging through the reaction liquid, escaped through the second port.

The 316 type sintered stainless steel sparger was of doughnut design with an OD of 5 in. while the ID hole was 1.5 in. The ultrasonic proble tip was inserted through the hole in the gas sparger and therefore sound waves flowed concurrently with the gas bubbles (average initial bubble diameter -5μ).

Included in the ultrasonic system was a power monitoring device which indicated peak envelope power (PWP), delivered from the sound generator to the head in the probe tip. The amount of acoustical power delivered was variable (0 to 300 watts) and controllable. Energy utilization for this unit has been reported by Sierka. (1)

Ozone was generated from commercial air delivered from cylinders at 15 psig and from pure oxygen tanks at 10 psig to the OREC ozonator model 03B1-0. Ozone concentrations in air ranged from 0.86% to 1.19% while from pure oxygen it was 2.20% to 2.40%.

COD, TOC, temperature (°C) and pH were measured on all liquid samples taken at 30 min intervals over the 2 hr ozonation period. Ozone gas samples from the ozonator and reactor effluent lines were measured at irregular intervals. COD and ozone gas concentrations data were obtained by employing standard methodologies. (2) TOC data were measured by directly injecting liquid samples (in triplicate) and standards into a Beckman Model 915 Total Organic Carbon Analyzer.

A series of oxygen uptake runs were performed in the reactor. Laboratory distilled water was placed in the column to a depth of 6.0 ft. Sodium bisulfite and cobalt chloride were added according to the 13th Edition of Standard Methods. (2) The water was discarded after companion runs were conducted (i.e., with and without sound) to prevent spurious results due to the build-up of salts or catalyst. Dissolved oxygen measurements were made with a Yellow Spring Dissolved Oxygen Meter model 57 placed at depths of 6-in., 12-in. and 55-in. above the gas sparger. The time to reach each 0.5 mg/l change in dissolved oxygen of the reactor contents was recorded and used to calculate the overall oxygen gas transfer coefficient (K_La).

A more detailed description of the experimental apparatus and methodologies has been reported. (3)

RESULTS AND DISCUSSION

The Effects Of Sound Waves On The Ozone Oxidation Of MUST UF And RO Permeates

*BIOSONIC IV

To provide data to derive kinetic equations for the design of ozone reactors employmeing sound waves as a catalyst and to improve gas mass transfer, a series of semi-batch experiments were carried out. Three experiments each were performed at constant VVM and variable ozone inlet gas concentrations and three experiments with variable VVM and constant ozone inlet gas concentrations with UF and RO permeates serving as the substrates. The following operating conditions were maintained in these experiments;

(1) a starting pH of 9.0 and uncontrolled during the reaction,
(2) ambient temperature operation (approximately 26 ± 1°C),
(3) a 6.0 foot water depth in the reactor, and
(4) A batch reaction time of 2.0 hours.

During the first three runs, conducted with UF permeate, an approximately constant ozone concentration of 1.0 weight percent (0.96% - 1.03%) in air was employed. The reaction gas was applied at three different VVM;s (0.1, 0.2, and 0.4).

Figure 2 is a semi-log plot showing the ratio of TOC/TOC_0 as a function of batch ozonation time, while Figure 3 displays the run data on a COD basis. In Figure 4 is graphed the ratio of ozone leaving the reactor to that entering at various batch reaction times.

From Figures 2 and 3 it is evident that, as VVM is increased, the rate and extent of the organic destruction is enhanced. After 2 hours of ozonation, the TOC remaining for the 0.1, 0.2, and 0.4 VVM runs was 88%, 70%, and 48% respectively while on a COD basis the corresponding percentage remaining at the end of these runs was 72%, 48%, and 22%. These results basically reflect the difference in the nature of the TOC and COD tests. Where the TOC test quantifies the loss of organic carbon by total oxidation to carbon dioxide, the COD test indicates the change in molecular chemistry of the dissolved substrate and measures the oxygen demand of the substance remaining. The COD test, in other words, gives indication of the partial oxidation of the dissolved organics in solution.

The ozone concentrations of the gas leaving the reactor was monitored at irregular times throughout the run. In Figure 4, it can be seen that all of the ozone applied to the reactor was utilized when the UF permeate was being oxidized by ozone at the 0.1 VVM condition. Thus, the run was totally mass transfer controlled and represents the major reason that only 12% of the TOC and 28% of the COD was oxidized. The average ultrasound PEP level for the experiment was at the 1.0 weight % ozone-in-air concentration, no residual ozone was found in the gas coming from the top of the reactor until 60 minutes of the run had been completed. From that time on, to the end of the run at 120 minutes, the effluent ozone gas concentration ranged between 34% and 46%. Finally, when the VVM was increased to 0.4, ozone first appeared in the effluent gas stream from the column at 10 minutes into the run. The increase in ozone concentration from 10 to 120 minutes was linear with batch reaction time, until at the termination of the run, 96% of the ozone gas admitted was found to be exiting the reactor.

The next series of three runs were performed at constant VVM (0.4) and variable ozone gas concentration. Ozone gas concentrations were 0.86%, 0.96%, and 2.21% by weight in air. The first two ozone concentrations were produced from air while the 2.21% ozone by weight was prepared from pure oxygen. The use of oxygen was necessitated by the fact that the ozonator could not produce an ozone concentration greater than 1.0% by weight when air was used as the feed gas, for the 0.4 VVM operating conditions. The object of these last three runs was to obtain a portion of the data required to derive a kinetic relationship and quantify the ozone partial pressure effect.

From Figures 5 and 6 it is highly evident that the partial pressure effect is highly significant in reducing the time required to produce a given organic quality effluent. Sierka (1), Gollan et al (4) and See et al (5) previously likewise demonstrated this effect on MUST wastewaters.

Consideration of Figure 7 however, shows that as ozone concentration is increased, the amount of ozone leaving the reactor likewise increases, thus operating and energy cost penalties are incurred unless this ozone can be utilized in the precontactor or collected and readmitted to subsequent reactor stages.

Utilizing the data plotted in Figures 2, 3, 5 and 6, a kinetic equation of the form:

$$-\frac{d(\mathrm{TOC})}{dt} = K_1 \; (\mathrm{VVM})^{R_1} \begin{pmatrix}\text{Inlet Ozone}\\ \text{Concentration}\end{pmatrix}^{S_1} \begin{pmatrix}\text{Organic}\\ \text{Concentration, TOC}\end{pmatrix} \qquad (1)$$

$$-\frac{d(\mathrm{COD})}{dt} = K_2 \; (\mathrm{VVM})^{R_2} \begin{pmatrix}\text{Inlet Ozone}\\ \text{Concentration}\end{pmatrix}^{S_2} \begin{pmatrix}\text{Organic}\\ \text{Concentration, COD}\end{pmatrix} \qquad (2)$$

where

R_1, R_2, S_1, S_2, = exponents

$K_{1,\,2}$ = a proportionality constant which is a function of system operating variables

VVM = volume gas applied/minute/volume reaction liquid

Inlet Ozone Concentration = weight percent ozone in carrier gas

was developed for the oxidation of UF and RO permeates by ozone in the presence of ultrasound.

Using log-log plotting techniques, the constants R_1, R_2, S_1, and S_2 were evaluated and for the case when synthetic UF permeate was the substrate, the following equations resulted.

$$-\frac{d(\mathrm{TOC})}{dt} = K_1 \; (\text{Inlet Ozone Concentration})^{1.4} \; (\mathrm{VVM})^{1.0} \; \mathrm{TOC} \qquad (3)$$

$$-\frac{d(\mathrm{COD})}{dt} = K_2 \; (\text{Inlet Ozone Concentration})^{1.0} \; (\mathrm{VVM})^{1.1} \; \mathrm{COD} \qquad (4)$$

For identically performed experiments employing synthetic RO permeate, the following values were obtained; (1) $R_1 = 3.0$ (2) $S_1 = 0.7$ (3) $R_2 = 0.8$ and (4) $S_2 = 1.4$.

Ultrasound Scale-up Experiments With UF Permeates

The benefits of sound inclusion into any ozone reactor are expected to come from improvement in gas transfer and decomposition of ozone to various free radicals. Sierka (1) has demonstrated with small laboratory size equipment that these benefits do exist.

For larger systems the question of scale-up procedure must be addressed. Today, there does not exist a sufficient body of technical expertise for reactor design employing ultrasonics to permit scale-up based on well defined scientific principles and therefore, an empirical approach must be used. Since the experiments performed during this research were limited to the use of a sound generator, operating basically at one frequency (20kHz) liquid height in the reactor was varied and maximum achievable sound power was employed in an attempt to characterize this effect.

For these experimental runs, four liquid heights (1.0, 2.0, 3.0, and 6.0 feet), which on a liquid volume basis corresponded to 5.55, 11.1, 16.0, and 33.3 liters of synthetic UF permeates, were ozonated at essentially constant ozone gas concentration (approximately 1.1% by weight ozone in air) and a constant superficial gas velocity of 2.39 feet per minute. Even though the superficial gas velocity was constant, the VVM's were variable because VVM is calculated to be the volume of gas per unit minute admitted to the reaction mass per volume of liquid.

The results of these experiments are summarized in Table 2. These data indicate a direct proportional increase in the mass of TOC removed with liquid height. Also it is evident that ozone utilization increased with liquid height. This can be seen in column 7 of Table 2 which is the ratio of ozone utilized in the reactor to the theoretical amount of ozone required to completely oxidize the amount of TOC listed in column 2. These data indicate that the theoretical limit of one was being approached when 6.0 ft of liquid was present in the reactor. Since a maximum of 6.5 ft is available for the reactor in the MUST system, this is a moot point. However, this approach could be applied to other ozone reactors not constrained in this manner.

The Effect Of Ultrasound On Oxygen Mass Transfer

It has been postulated by Lee and See (6) that in the initial stages of ozone oxidation of MUST RO permeates (and therefore for UF permeate as well) the reaction is controlled by the transfer of ozone from the bulk gas to the bulk liquid phase. They maintain that mass transfer would be aided by decreasing bubble size, increasing superficial gas velocity and ozone partial pressure.

Bubble size, for experiments performed by USAMBRDL and the authors, is already minimized since the sparger produces 5μ-size bubbles. The maximum ozone partial pressures obtainable are limited since the ozone must be produced from air in MUST field operations. The state of current technology is such that 1.0 weight % ozone-in-air is the upper concentration limit of production capability. Superficial gas velocity in the ozone reactor is related to the ozone production capability. If the gas mass flow rate through the ozonator is increased, the ozone concentration will decrease and thus the driving force to gas transport will be reduced.

Because of the particular parameter association in the ozonator and reactor systems, other measures must be applied if gas mass transfer is to be enhanced. The application of sound waves to the reaction mass is one such available alternative.

To evaluate the effect that ultrasound had on mass transfer, a series of oxygen uptake runs were performed in the reactor at 0.1, 0.2, and

0.4 VVM. Overall oxygen transfer coefficients (K_La) were measured at the 6.0, 12.0 and 55.0 inch levels from the gas sparger surface.

From Table 3 it can be seen that K_La, at any given position in the column, increased with increased VVM regardless of the presence or absence of sound waves. However, the presence of sound waves in the reactor always yielded an improvement in oxygen mass transfer over the companion no sound condition. The range of improvement was between 1% and 48%.

These latter results were not unexpected. Dahi (7) had experimented with the effect of 20kHz ultrasonic waves on the transfer of oxygen to distilled water and using a 38 mm ultrasonic probe and 160 watts of power, reported increases in K_La of between 15-45% when oxygenating 1.0 liters of distilled water by means of 0.4 mm i.d. hypodermic needle.

The Effect Of Ultrasound On The Air-stripping Of UF Permeate

McCarthy (8) et al investigated the effectiveness of air-stripping to remove dissolved organics contained in RO permeates from MUST laboratories. They developed a correlation between a first-order stripping rate constant and the overall mass transfer coefficient.

The success of this mathematical model led to further experimentation with MUST hospital UF permeate serving as the substrate. The experimental conditions and results for this work are listed in Table 4. From these experiments it was concluded that air-stripping of UF permeates was of little value. (8) However, experiments were carried out by the authors to determine if air-stripping aided by ultrasound could reverse this conclusion.

A 33.3-1 batch of UF permeate at pH-8.9 was preheated to 45°C and air-stripped at a VVM of 0.69 for three hours. Table 5 lists the data collected during the run.

From these figures it is evident that air-stripping was not particularly successful. However, it can be stated that the rates of COD and TOC removal in the presence of ultrasound did exceed those obtained by USAMBRDL. This increase can be attributed to the enhanced mass transport properties of the air-sound system.

The results indicated however, that practically speaking, air-stripping even in the presence of sound would preclude the use of this concept in the field due to the unreasonably high hydraulic detention times required.

SUMMARY

Organic oxidation rates for synthetic UF and RO permeates, as measured by COD and TOC destruction, were increased by either increasing VVM at fixed ozone gas concentration or increasing inlet ozone concentration at a fixed VVM in a reactor employing sound waves. The benefits due to increasing ozone partial pressure were somewhat negated by the increased amounts of unreacted ozone exiting the reactor. It was also shown that ozone utilization can be increased by increasing the liquid height in the reactor through which the ozone was being bubbled. A mathematical model was developed to relate organic oxidation rates to reactor inlet ozone concentration, VVM and organic concentration.

The positive effect of ultrasound on the transfer of oxygen to distilled water was demonstrated. K_La increased directly with increases in VVM and position above the gas sparger. The improvement ranged from 1% to 48% over the no-sound condition.

This result partially explains the improvement in the air-stripping rates of organics from UF permeates when sound waves were employed.

ACKNOWLEDGEMENTS

This work was supported by the US Army Medical Bioengineering Research and Development Laboratory under Contract No. DAMD-17-77-C-7031.

REFERENCES

1. Sierka, R. A., "Mass Transfer and Reaction Rate Studies of Ozonated MUST Wastewaters in the Presence of Sound Waves", US Army Medical Research and Development Command, Contract DAMD-17-76-C-6057, Final Report (1976).
2. Standard Methods for the Examination Water and Wastewater, Thirteenth Edition, American Public Health Association, Washington, D.C., 1971.
3. Sierka, R. A., "Effects of Ultrasound on MUST Hospital Composite Wastewaters", US Army Medical Research and Development Command, Contract DAMD-17-77-C-7031, Final Report, Nov. 1977.
4. Gollan, A. Z., K. J. McNulty, R. L. Goldsmith, M. H. Kleper and D. C. Grant, "Evaluation of Membrane Separation Processes, Carbon Adsorption and Ozonation for Treatment of MUST Hospital Wastes", DAMD-17-74-C-4066, Final Report (1976).
5. See, G. G., P. Y. Yang, and K. K. Kacholia, "Research and Development of an Ozone Contactor System", US Army Medical Research and Development Command, Contract DAMD-17-76-C-6041, Final Report (1976).
6. Lee, M., and G. See, "Ozone Technology Presentation", Life Systems, Inc., Engineering Report 314-35, Cleveland, OH (1977).
7. Dahi, E., "Physiochemical Aspects of Disinfection of Water by Means of Ultrasound and Ozone", Water Research, 10, 677, 1976.

TABLE 1

ORGANIC CHEMICAL COMPOSITION OF HOSPITAL COMPOSITE
ULTRAFILTRATION PERMEATES AND REVERSE OSMOSIS PERMEATES

	UF PERMEATE	RO PERMEATE
methanol	29.8 µl/l	16 µl/l
acetone	6.3 µl/l	5 µl/l
acetic acid	3.4 µl/l	2.8 µl/l
diethyl ether	0.6 µl/l	0.1 µl/l
N,N-Diethyl-m-toluamide	0.8 mg/l	0.2 µl/l
ethanol	0.5 µl/l	0.5 µl/l
oleic acid	0.5 µl/l	0.1 µl/l
phenol	1.3 mg/l	0.4 mg/l
urea	18.0 mg/l	12.0 mg/l
Kodak X-Omat Developer	942 µl/l	283 µl/l*
Kodak X-Omat Fixer	942 µl/l	283 µl/l*

---

* Assumed R.O. membrane 70% rejection based on Du Pont B-10 separation characteristics and an estimated 2:1 ratio for hydroquinone: acetic acid content which forms the major organic portion of the fixer/developer.

TABLE 2

OZONE UTILIZATION IN VARIABLE HEIGHT EXPERIMENT WITH SYNTHETIC UF PERMEATE

[1] Reactor Liquid Volume (l)	[2] TOC Removed in 1 Hr (mg)[a]	[3] Theoretical O_3 Required (mg)[b]	[4] Total O_3 Applied (mg)[c]	[5] Total O_3 Unreacted (mg)[d]	[6] Total O_3 Utilized in Reaction (mg)[e]	[7] $\frac{O_3 \text{ Utilized}}{\text{Theoretical } O_3 \text{ Required}}$ (mg/mg)[f]
5.55	111	888.0	11,767	8,781	2,986	3.36
11.10	222	1,760.0	12,037	7,459	4,578	2.60
16.00	320	2,560.0	12,308	6,510	5,798	2.26
33.33	649	5,196.0	10,955	2,366	8,589	1.65

Note:

a. TOC in (mg/ℓ) - $\text{TOC}_{60 \text{ minutes}}$ (mg/ℓ) x volume (l) in Column 1

b. Theoretical ozone demand is calculated to be 8 times the mass of TOC removed. This assumes that only nascent oxygen is capable of totally ozidizing the TOC to CO_2.

c. Ozone mass flow rate (mg/min) from ozonator x 60 minutes

d. Integrated average over the run based on reactor effluent gas analysis

e. Column 4 - Column 5

f. Column 6/Column 3

TABLE 3

OVERALL MASS TRANSFER COEFFICIENT (K_La) MEASUREMENTS FOR DISTILLED WATER IN A 6" DIAMETER COLUMN REACTOR IN THE PRESENCE AND ABSENCE OF SOUND WAVES

VVM	0.1	0.2	0.4
Dissolved Oxygen Probe Height (in.) From Sparger	6	6	6
K_La (HR.$^{-1}$)			
PEP -(0) Watts	15.8	26.9	35.4
PEP -(110) Watts	20.9	29.5	42.7
VVM	**0.1**	**0.2**	**0.4**
Dissolved Oxygen Probe Height (in.) From Sparger	12	12	12
K_La (HR.$^{-1}$)			
PEP (0) Watts	36.1	37.2	95.8
PEP (100) Watts	49.6	55.1	112.1
VVM	**0.1**	**-**	**0.4**
Dissolved Oxygen Probe Height (in.) From Sparger	55	-	55
K_La (HR.$^{-1}$)			
PEP (0)	46.8	-	106.7
PEP (130)	47.1	-	121.6

Liquid Depth - 6.0 ft.

TABLE 4

U.S. ARMY MEDICAL BIOENGINEERING R & D LABORATORY UF STRIPPING RUN

REACTOR CONDITIONS

Gas stripping bottle in a constant temperature bath

Temperature - 49°C
Wastewater volume - 500 ml
VVM - 0.8
Gas type - Air

TEST RESULTS

TIME (MIN)	TOC (mg/ℓ)	COD (mg/ℓ)
0	62.2	259
15	60.8	261
60	62.3	264
120	61.4	261

TABLE 5

ULTRAFILTRATION PERMEATE STRIPPING IN THE PRESSURE OF ULTRASOUND

REACTION TIME (MIN.)	TEMP (°C)	COD (mg/ℓ)	TOC (mg/ℓ)	PEP (WATTS)
0	45.0	254.	55	170
60	44.0	232.	53	170
120	39.0	229.	49	165

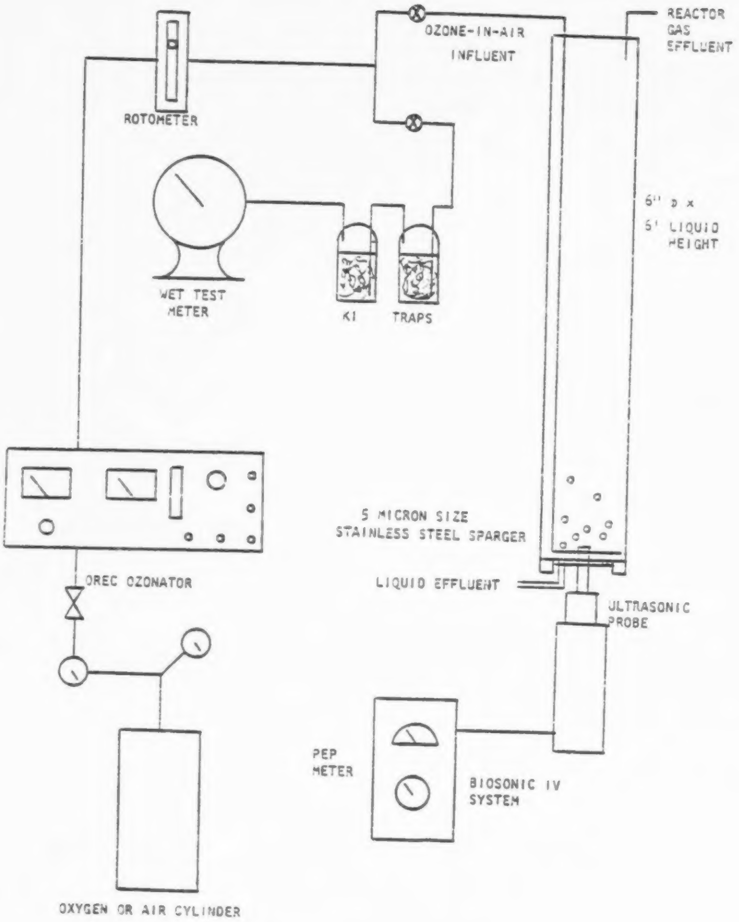


FIGURE 1. EXPERIMENTAL APPARATUS.

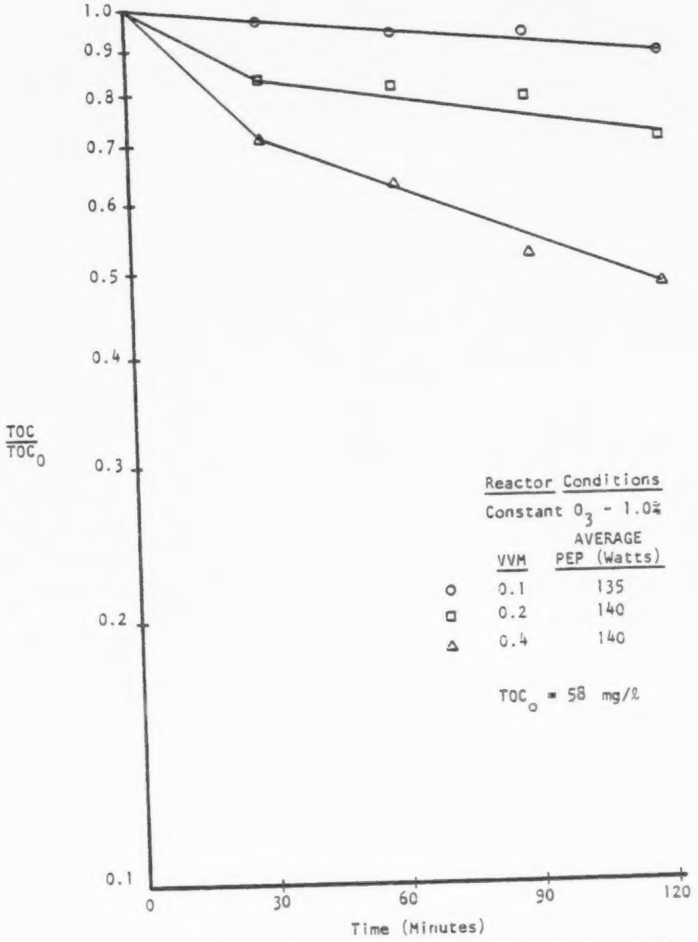


FIGURE 2. EFFECT OF GAS FLOW RATE ON TOC REMOVAL FROM ULTRAFILTRATION PERMEATES AT CONSTANT OZONE CONCENTRATION AND VARIABLE VVM.

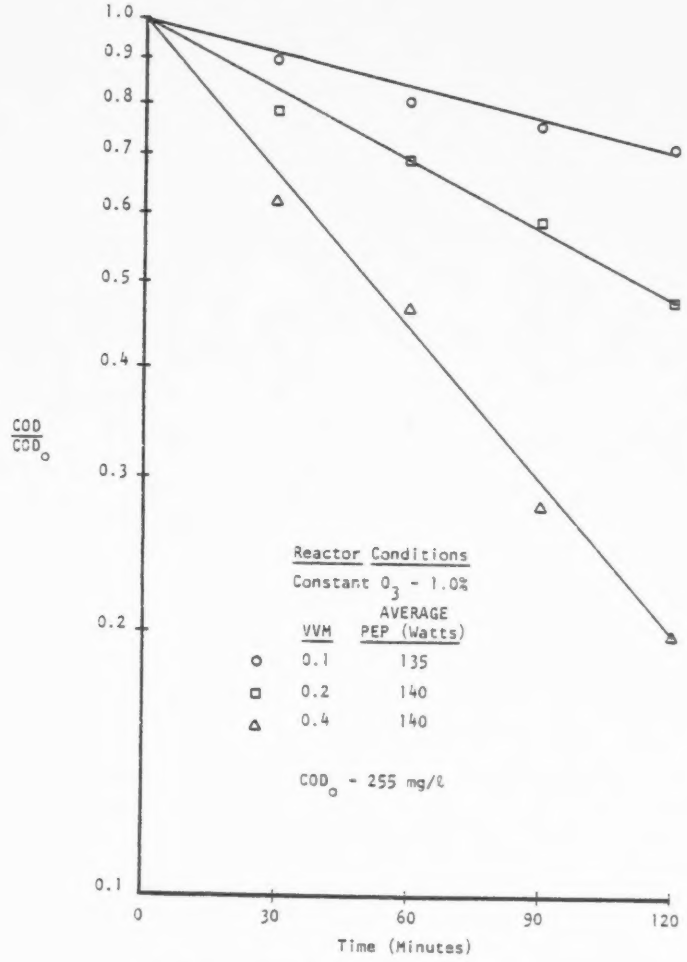


FIGURE 3. EFFECT OF GAS FLOW RATE ON COD REMOVAL FROM ULTRAFILTRATION PERMEATES AT CONSTANT OZONE CONCENTRATION AND VARIABLE VVM.

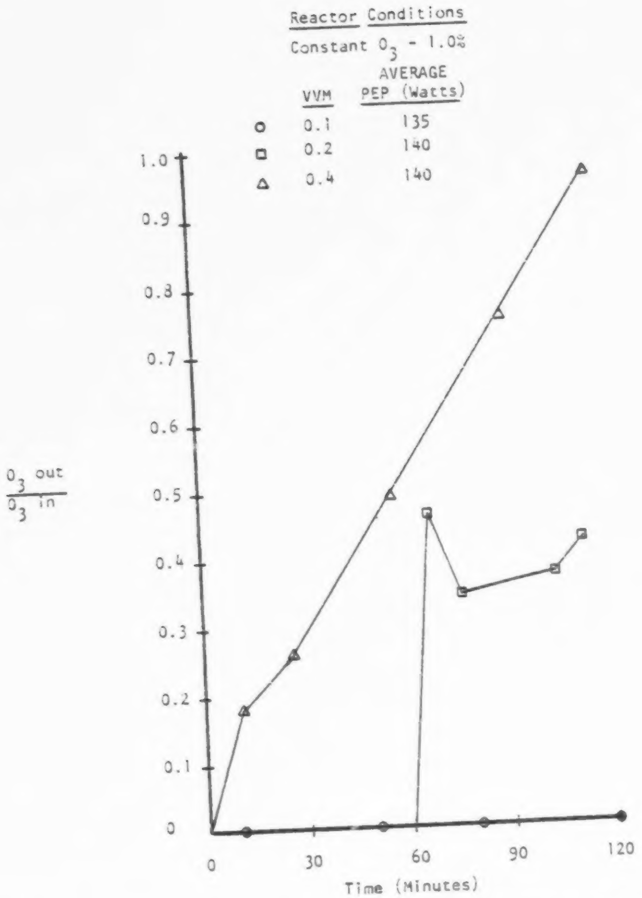


FIGURE 4. THE RATIO OF REACTOR OUTLET TO INLET OZONE CONCENTRATION AT VARIOUS REACTION TIMES DURING THE OXIDATION OF ULTRAFILTRATION PERMEATES AT CONSTANT OZONE CONCENTRATION AND VARIABLE VVM.

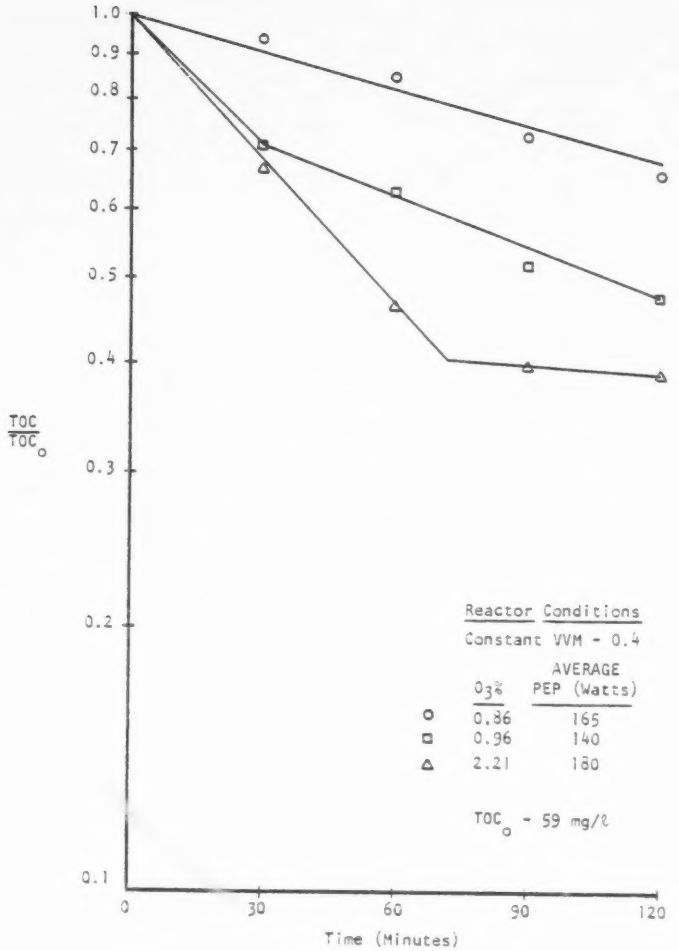


FIGURE 5. EFFECT OF OZONE CONCENTRATION ON TOC REMOVAL FROM ULTRAFILTRATION PERMEATES AT CONSTANT VVM AND VARIABLE OZONE CONCENTRATION.

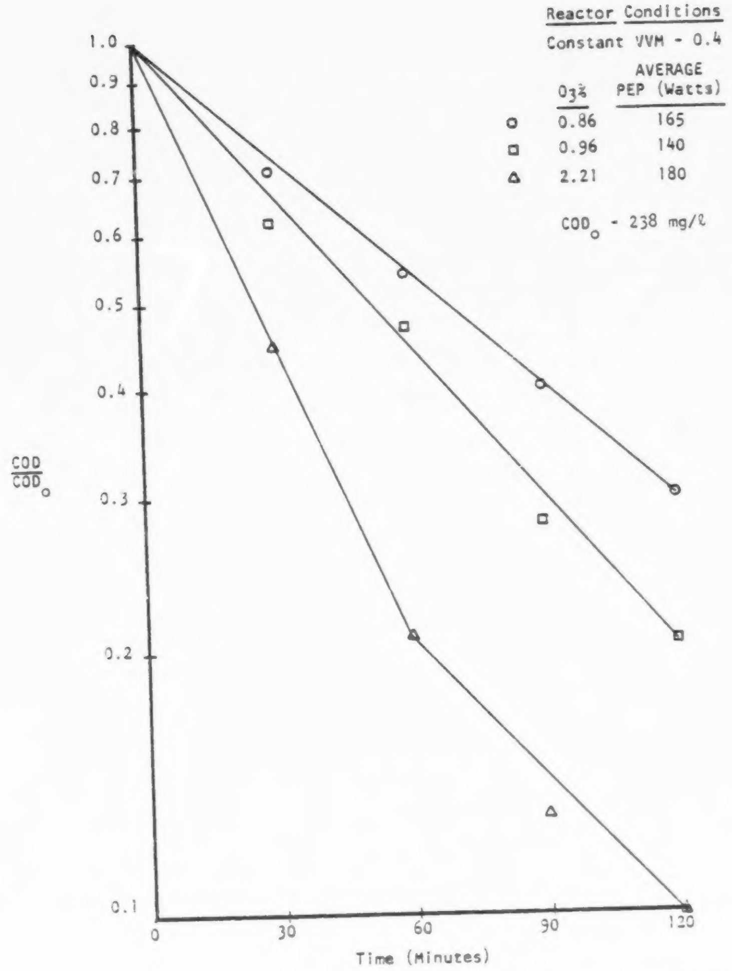


FIGURE 6. EFFECT OF OZONE CONCENTRATION ON COD REMOVAL FROM ULTRAFILTRATION PERMEATES AT CONSTANT VVM AND VARIABLE OZONE CONCENTRATION.

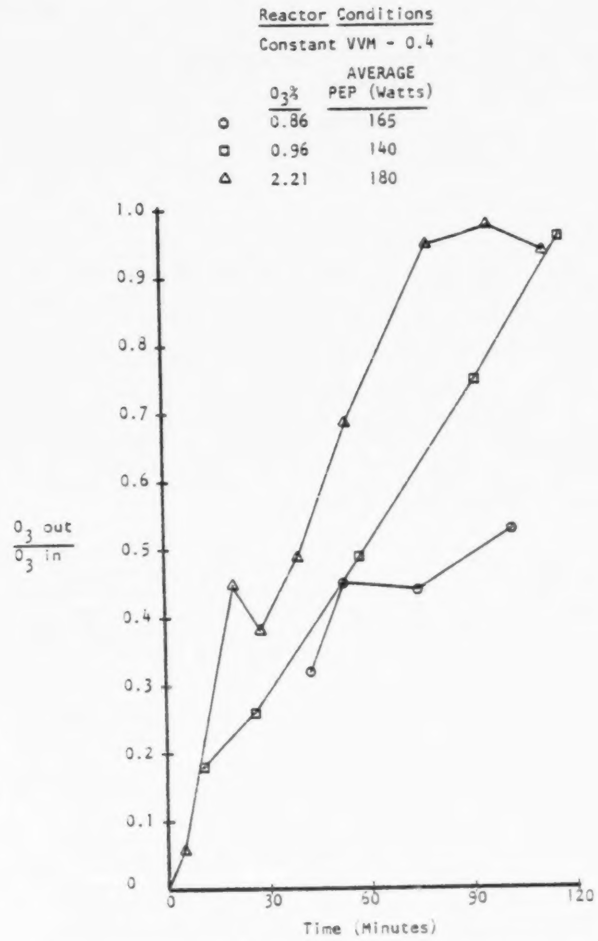


FIGURE 7. THE RATIO OF REACTOR OUTLET TO INLET OZONE CONCENTRATION AT VARIOUS REACTION TIMES DURING THE OXIDATION OF ULTRAFILTRATION PERMEATES.

RENOVATION OF MUNICIPAL WASTEWATER BY REVERSE OSMOSIS: A Case Study

C. K. Wojcik
University of Petroleum & Minerals
Dhahran, Saudi Arabia

J. G. Lopez
California Department of Water Resources
Sacramento, California

J. W. McCutchan
University of California
Los Angeles, California

INTRODUCTION

With continuing trend toward greater centralization of population in metropolitan areas, coupled with limited availability of fresh water, the need for renovation of municipal wastewaters becomes acute.

To protect the public, renovated wastewaters must meet certain health standards before they can be used for irrigation, recreation, or even human consumption. It is therefore, important that such waters be relatively free of microbiological and chemical contaminants. There are several ways in which such purification of water can be accomplished. The objective of this research program was to evaluate the effectiveness of the reverse osmosis process in removing the contaminants with minimum of pretreatment of feedwater.

The Las Gallinas Valley Sanitary District, Marin County, California, operates a sewage treatment plant a few miles north of San Rafael, California. In 1975, the District, in cooperation with the California Department of Water Resources (DWR) and the University of California at Los Angeles, (UCLA), installed a 160-tube reverse osmosis unit for a pilot study on application of this process to renovation of municipal wastewater.

In 1977, School of Public Health, University of California at Berkeley, joined the program under contract to DWR to perform biological and chemical studies (1).*

SEWAGE TREATMENT PLANT

Raw sewage is delivered to the plant through a 30 in. pipe. The amount received is usually about 2.5 mgd but it may go as high as 10 mgd during a rainy season. The sewage first passes through a shredding device (see Fig. 1), then it goes to a sump beneath the pump station. Two centrifugal pumps, driven by electric motors, pump the sewage to two clarifiers. One pump motor is a two-speed motor, 1170/870 rpm, 20/11.3 hp, the other is a single-speed motor, 870 rpm, 10 hp.

The clarifiers are large cylindrical tanks of about 65 ft. diameter with a slightly conical bottom. The depth of the clarifiers is 6 ft. at outside walls, and 7 ft. in the center. The main feature of each clarifier is a rotating structure that carries two long and two short lower arms that scrape the sludge from the bottom of the clarifier, and one upper arm that skims the surface of the wastewater. The

*Numbers in brackets designate references at end of paper.

structure rotates about the center column at the rate of 1 rev/45 min. The wastewater, delivered at the bottom of a clarifier, moves upward through and over the column into the clarifier's basin. Sludge collects at the bottom, foam and other floating matters are skimmed from the surface. The wastewater is diverted to clairfier's outer channel and, then, it is pumped to two trickling filters. Sludge is pumped to a thickener.

The trickling filter #1 is a large (110 ft. in diameter) and shallow (3 ft. deep) cylindrical basin filled with rocks. The bottom of the filter is slightly conical, its lowest point is at the center. Two arms spread across the full length of the diameter and form one structure that is pivoted about a center column and is free to rotate about it. The arms carry nozzles mounted in such a way that the reactive force of the out-flowing water causes them to rotate in CW direction. The rate of rotation depends on the rate of flow of water through the nozzles. As the arms rotate, the wastewater is sprinkled over the rocks that are covered with bacteria slime. The organic material is degraded by the bacteria as the water trickles to the collecting channel at the bottom of the filter. From the trickling filter #1, the water goes back to the clarifiers. This creates a partial closed cycle; part of the output of the clarifiers, equal to the amount of raw sewage coming to the plant, is directed to the trickling filter #2. The water from clarifiers is pumped to the trickling filters by means of centrifugal pumps driven by two-speed (875/695 rpm) electric motors of 15/9.6 hp.

The trickling filter #2 is of similar construction as #1, only larger (140 ft. in diameter) and it has four arms. From this filter, under the gravity force, water flows to clarifier #3 (final) whose capacity is twice that of clarifier #1 or #2. Part of the water from this clarifier goes back to the trickling filter #2, another part goes through chlorine injection process and multi-media filter to the R.O. unit and to a 20,000 gal. storage tank (water used for landscaping), the third part is chlorinated and diverted to a contact chamber (to achieve residence time of 1.5 hours), then it is discharged into a creek and subsequently to San Pablo Bay. Before entering the creek the water is stripped of chlorine by means of sulfur dioxide.

Sludge from clarifiers #1 and #2 is pumped to a thickener that looks like a clarifier but is smaller in diameter (30 ft) and higher (10 ft). Thickened sludge collects at the bottom, from there it is pumped to a shredder, and then to digester #1. This digester is a cylindrical tank, 60 ft. in diameter and 26 ft. high. The sludge there is mixed and heated to about 95°F (to optimize the digestive process). The heating is done by passing some of the sludge through a heat exchanger which is heated by methane gas collected in the two digesters. Under the gravity force, sludge flows to digester #2, 50 ft. in diameter and 21 ft. high. There, the sludge is neither heated nor mixed. Both digesters produce more methane gas than is needed for heating sludge; the excess of unscrubbed gas is burnt off. 10,000 to 12,000 gallons of sludge is processed this way a day. The sludge from the digester #2 contains 5% of solids. Their amount is increased to 22% by means of centrifuge. Concentrated sludge is collected in a sump and then hauled away in trucks. Water from the digesters and from the centrifuge is pumped to a pond.

R O UNIT

Part of the processed wastewater from the clarifier #3 is diverted to a multi-media filter and then to two storage tanks, as shown in Figure 2. One tank, of capacity 2,000 gal., is used for storing the water for the R O unit; the second tank, of capacity 20,000 gal. for water that is used for landscape irrigation. When the 20,000 gal. tank fills up, the excess water goes to an underground Nike Missile vault of 300,000 gal. capacity. In case of shortage, the R O unit has the first priority on filtered water.

In the first phase of this program, water used for irrigation was chlorinated while the water for the R O unit was delivered unchlorinated. At present, both waters are chlorinated. The chlorine is injected just before the multi-media filter. The concentration of the chlorine in the water is held at 2.0 ppm level.

Water for the R O unit passes through micron filter, then it is pumped to a 160-tube assembly, as shown in Figure 3. The micron filter, manufactured by Cuno Engineering Corporation, uses 30μ, polyester, cartridges. The pump is a positive displacement pump, plunger type, horizontal triplex, manufactured by Worthington, Model KBF, 12.7 gpm displacement. It is driven by a 15 hp U.S. Varidrive motor. The pressure in the system, 600 psi. is maintained by means of a pressure regulator (Grove Flexflo). The bypass valve, shown in the diagram, is normally closed; it is open during the membrane cleaning operation. The rate of flow of the feedwater, entering the tube assembly, is governed by varidrive setting. In the initial phase of the program it was held at 10 gpm; later on, at 5 gpm to increase water recovery. The tube assembly is shown in Figure 4. The 160 tubes are connected in series and arranged in five tiers. Individual tube modules are of typical UCLA design described in several publications (2,3). A module consists of a tube-shaped cellulose acetate (CA) semipermeable membrane, wrapped in three layers of dacron cloth, and enclosed in a perforated titanium tube of 1-inch outside diameter. Each tubular module is 10 ft. long and has 2.24 sq. ft. of effective membrane surface area. The membranes were cast at the Firebaugh Facility (3) from a cellulose acetate solution using Eastman Chemical Company formulation, E-400-25. They were cured at 88-90°C temperature. The desalination process takes place as the feedwater flows through the 160-tube assembly in a single pass. The desalted (product) water exits from the performations along the tubes, falls on the corrugated fiberglass sheets placed below the tubes, and is collected in troughs mounted at the end and below the respective corrugated sheets. The spacing of tubes is the same as the pitch of corruations, thus, product water from each tube flows in its own groove to an outlet located at the end of the sheet. This allows to take water samples from each tube individually and, thus, to measure performance of each membrane.

MAINTENANCE OF THE R O UNIT

Since the feedwater contains a relatively large amount of organic matter, fouling of membranes becomes a major problem in operating the system. To reduce it, cleaning of membrane was introduced.

Two types of cleaning are performed: chemical and mechanical. Chemical cleaning is done once a week, on Mondays, mechanical cleaning is done twice a week, on Mondays and Thursdays. One mechanical cleaning, on Mondays, follows immediately the chemical cleaning. For both cleanings, normal operation of the R O system must be interrupted. For chemical cleaning this interruption amounts to about three hours, for mechanical cleaning about one hour.

The chemical cleaning is done by means of citric acid and BIZ solution. Preparation of the solution is as follows: 250 g of citric acid and 500 g of BIZ are dissolved and mixed in 50 gallons of product water that is placed in one barrel. Same amounts of citric acid and BIZ are dissolved in the same amount of water in the second barrel. The solution from the first barrel is circulated through the R O tubes for one hour. Then, solution from the second barrel is circulated the same way for another hour.

Mechanical cleaning is accomplished by means of sponge balls (polyurethane, about 1-1/2 inch cube in undeformed state). Sponge balls are inserted into the tube assembly, at the inlet, at one minute intervals. Ten balls are used in this process. The outlet is connected to a bucket placed near a drain in the floor. Circulating feedwater pushes the sponge balls through the tubes cleaning the membranes in the process. The balls are collected in the bucket. It takes 15 to 17 minutes for a ball to pass through all 160 tubes. The last ball used is a colored ball to mark the end of cleaning process.

Recently, an automatic sponge-ball cleaning system has been installed. In this system, direction of flow of the feedwater is changed periodically. Any time such change takes place, a sponge ball is automatically inserted into the stream. No shutdown of the unit is required here. At present, the system is in experimental stage; efforts are made to select best material and shape for sponge ball and to establish the optimal frequency of the flow reversal.

High contamination of the feedwater by organic matter necessitates frequent changes of cartridges in the micron filter. They are changed every week. To keep the cost of this operation down, the cartridges are washed and used again later on.

The performance of each membrane is checked periodically. Twice a week, samples of product water are collected from each membrane and checked for salinity and flux. Any membrane, whose product water has TDS reading of 500 or more (feedwater salinity is about 1,200 ppm) or the flux drops below 5 gpfd, is replaced. During the period, May 1977 through June 1978, 116 membranes were replaced mostly due to high TDS reading, 15 membranes were ruptured during the sponge-ball cleaning.

R O UNIT PERFORMANCE

Because of high amount of contaminants in the feedwater, it was expected that the performance of the RO unit will be well below its possible optimum. The main problem encountered, during the unit's operation, was that of fouling of membranes leading to a rather fast decline of water recovery. Chemical and mechanical cleaning of membranes corrected this situation on at least temporary basis.

To keep record of system's performance, readings were taken daily of the following:

- Operating pressure
- Indicated flow rate
- Hour meter
- Power meter
- Product water meter

Also, measurements were taken of the following quantities:

- TDS of feedwater
- TDS of brine
- TDS of product water
- Flow rate of product water

Amongst the measured quantities, the flow rate and TDS of the feed and product waters are the most important ones. Their values are plotted in Figure 5-a for a selected period of time of 28 days (4 weeks). This period, March 20 through April 17 of 1978, was found to be a typical one in terms of system performance. The plots there are characterized by sharp discontinuities in the flow rate, Q, and the salinity, TDS, of the product water, representing the differences in readings taken before and after cleaning of membranes. Chemical and mechanical (sponge-ball) cleaning took place on 0, 7, 14 and 21st day, mechanical cleaning only, on 3, 10, 17 and 24th day of the period.

On the basis of these and other measured quantities, certain indices of performance were computed and are plotted in Figure 5-b. They are: salt rejection, C, water recovery, R, and power consumed, E. The first two quantities are expressed in percents, the third in kwh per 1000 gallons of product water. Since after cleaning of membranes, water recovery goes up while salt rejection drops down and both of these quantities are the main indices of system's performance, one may consider a single index that is a product of the two quantities, CR/100, also measured in percents. Values of this index are also plotted in Figure 5-b. They decrease with time, after cleaning of membranes, but not as rapidly as those of the water recovery alone.

Graphs, shown in Figure 5-a and 5-b, relate to the overall system performance. Since such performance depends on individual performances of the membranes, measurements were taken of TDS and of the flow rate of the product water from each tubular membrane. On the basis of these measurements, the water flux, F, in gallons per square foot per day (GFD), and the salt rejection, C, were computed for each membrane. Distribution of the values of these quantities amongst the membranes are shown in Figures 6 and 7. In both of these figures, graphs (a) illustrate the distribution of values measured right after chemical and sponge-ball cleaning; graphs (b) represent the distributions of values measured the following day, and graphs (c) are for measurements taken three days after cleaning. In all these graphs, the ordinate, N, represents the number of membranes whose values of flux or salt rejection fall within a considered interval. Right after cleaning of membranes, the values of flux vary widely. This variation decreases drastically with time, however, the average value of flux decreases rapidly too. Salt rejection on the other hand, is more stable; its variation is not nearly as great and its average value increases somewhat with time after cleaning.

The main objective of this research program was to evaluate the performance of the R O unit in renovating wastewater with minimum of pretreatment applied to the process. During the two-year study, the unit performed well considering large amounts of micro-biological and chemical impurities in the feedwater. The downtime for cleaning of membranes amounted to about 5 hours per week. Other interruptions in operation of the system were mostly due to external causes, such as power failures or malfunctioning of the multi-media filter pump.

The product water from the R O unit is used for irrigation of landscapeed areas, and as such, it must meet certain health standards based on the bacteria count. In the first phase of the program, there was no chlorination of the feedwater and the R O unit removed, on average, 99.95% of coliform bacteria and 85.53% of coliphage viruses (typical wastewater microorganisms), (1). This removal, combined with subsequent chlorination, gave the product water that met the prescribed health standards.

In the second phase of the program the feedwater was chlorinated at the 2.0 ppm level. The chlorination alone eliminated 99.998% of coliform bacteria and 98.255% of coliphage viruses. The R O system reduced the remaining coliforms by 80% and coliphages by 99.95%.

Addition of the sulfuric acid, in the third and last phase of the program, improved further the quality of the product water.

While chlorination is effective in destroying bacteria, it also causes formation of trihalomethanes when the chlorinated water is stored for days. This study has shown that the R O system has little effect on removal of these compounds.

The biological and chemical studies were performed by R. C. Cooper and M. G. Richard, School of Public Health, University of California at Berkeley, under contract to California Department of Water Resources, (1).

ACKNOWLEDGEMENTS

The authors are indebted to David L. Johnson of the Marin County Water District and Tom O'Haire of the Las Gallinas Valley Sanitary District for their contributions to the success of this program.

REFERENCES

1. Copper, R.C. & Richard, M.G. The Fate of Viruses, Bacteria, and Chlorination Byproducts in the Reverse Osmosis Process. Final Report (in preparation), School of Public Health, University of California at Berkeley (1978).

2. Johnson, J.S.; McCutchan, J.W. & Bennion, D.N. Three and One-Half Years Experience with Reverse Osmosis at Coalinga, California. UCLA Report No. 69-45 (July 1969).

3. Agricultural Wastewater Desalination by Reverse Osmosis. California Department of Water Resources Bulletin 196-76 (October 1976).

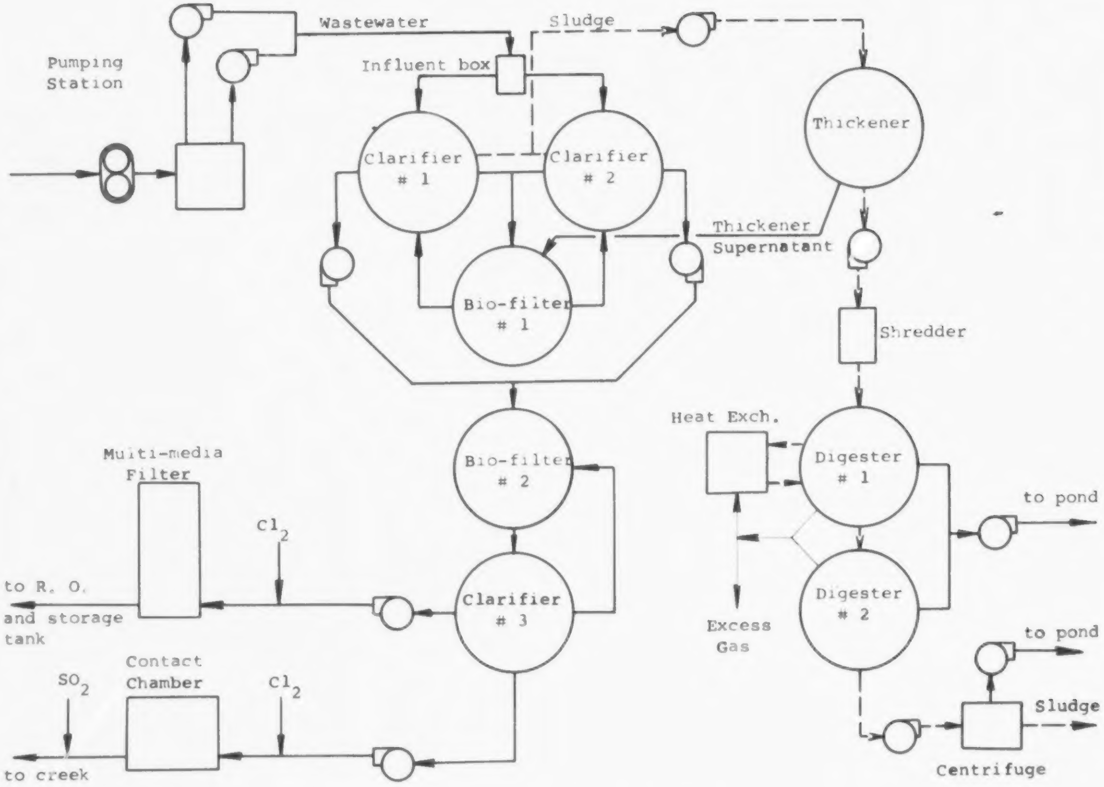


Figure 1. Las Gallinas Valley Sanitary District Wastewater Treatment Plant.

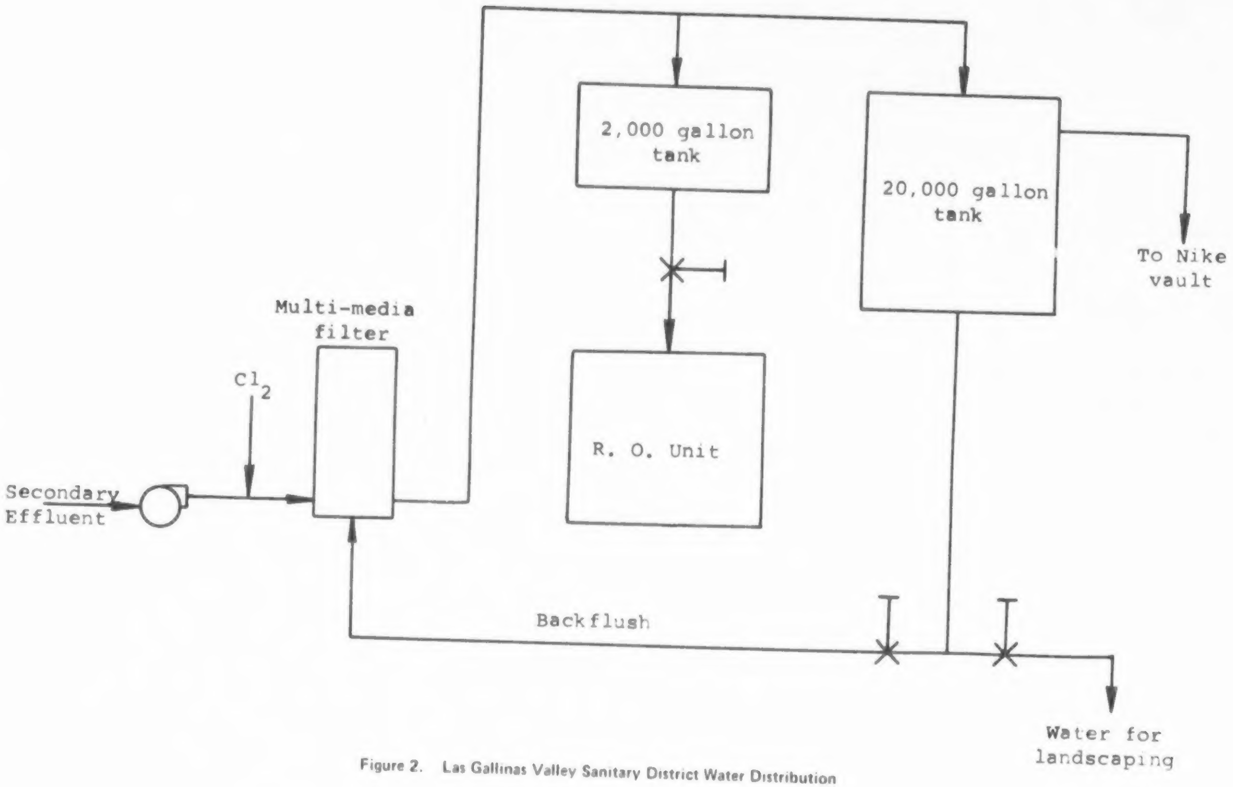


Figure 2. Las Gallinas Valley Sanitary District Water Distribution

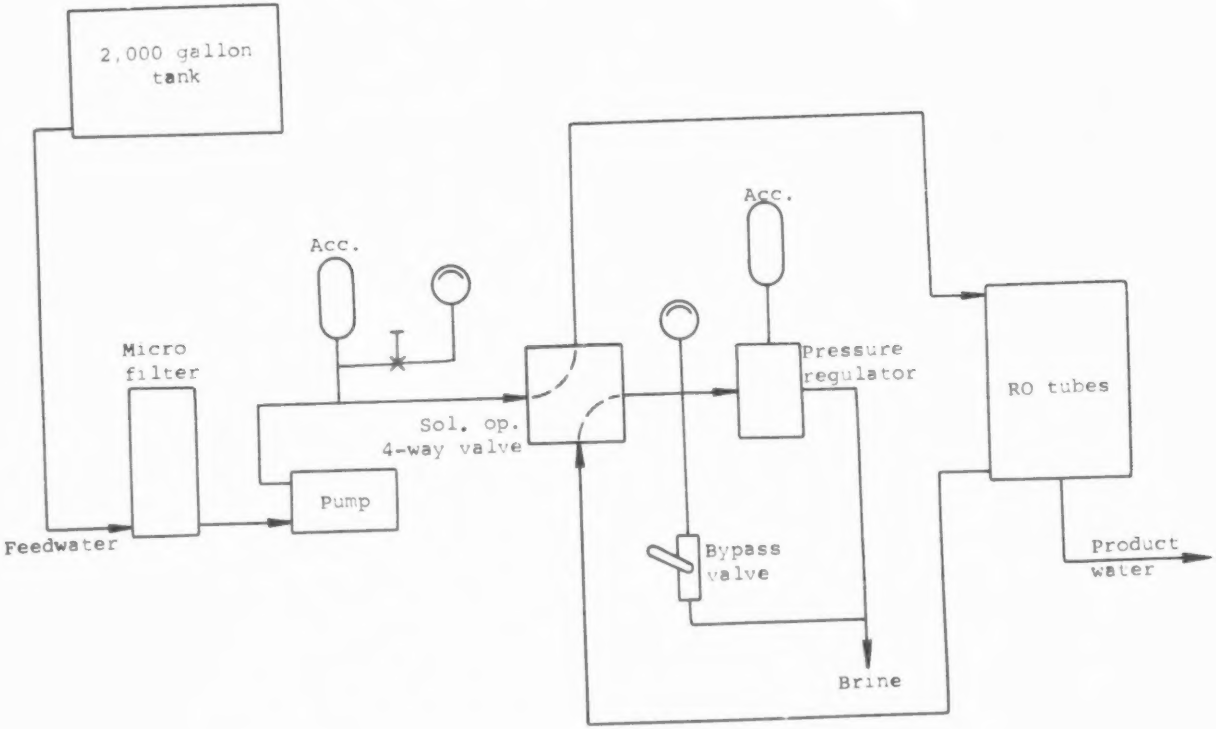


Figure 3. Reverse Osmosis Unit - Flow Diagram.

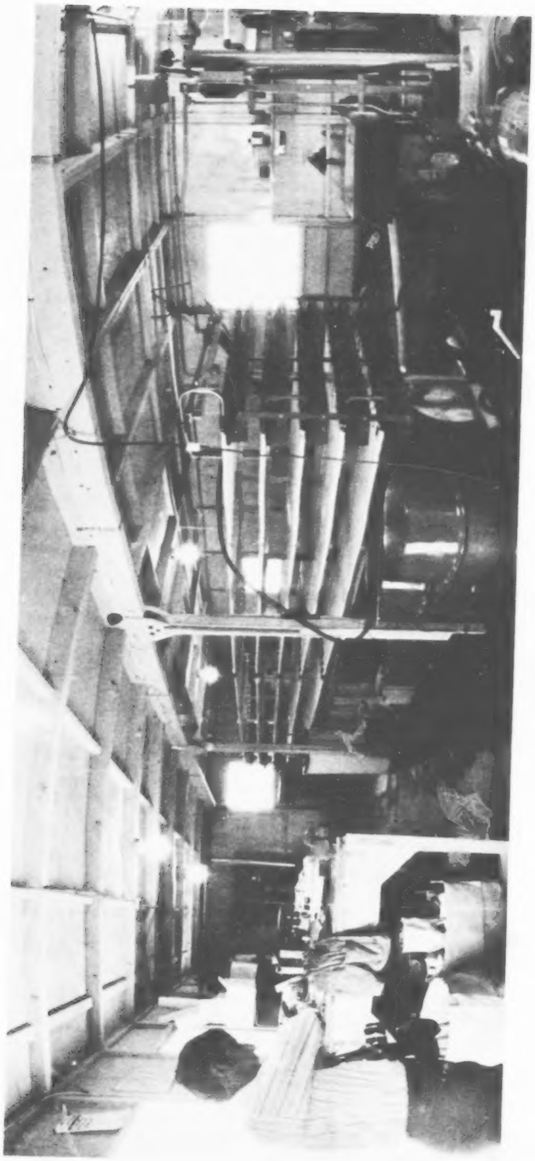
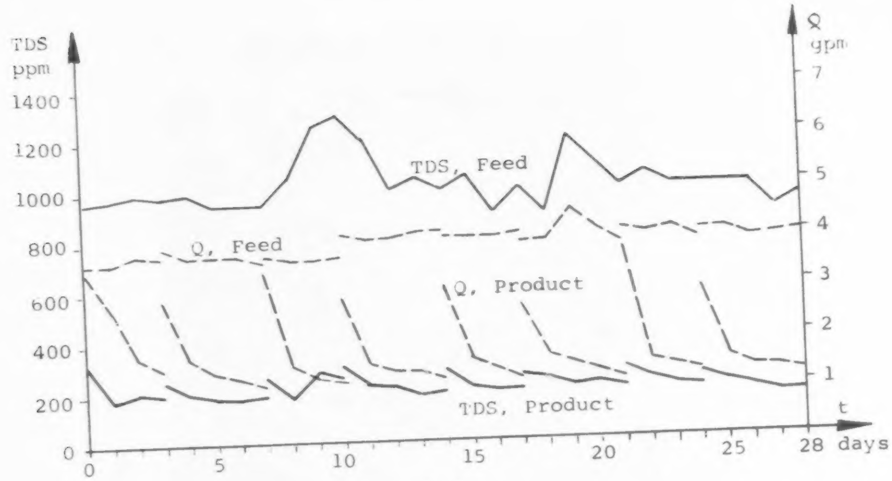
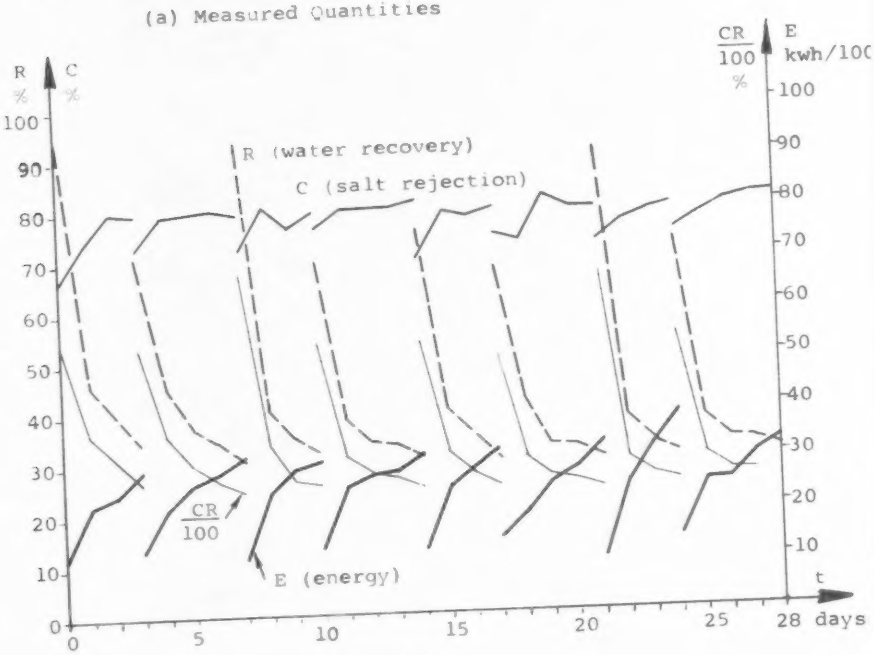


Figure 4. 160 Tubular Reverse Osmosis Assemblies.

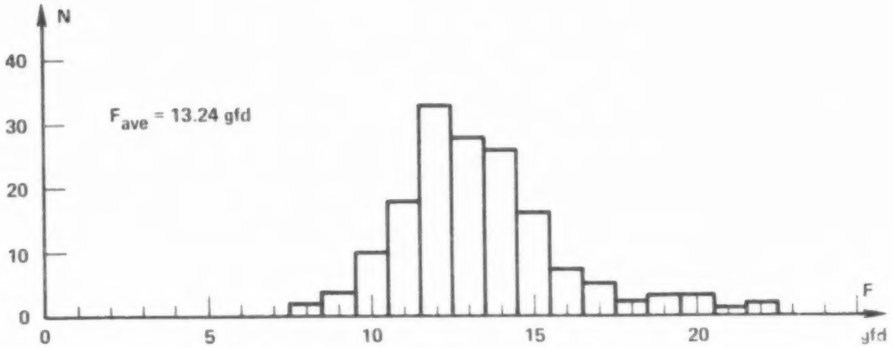


(a) Measured Quantities

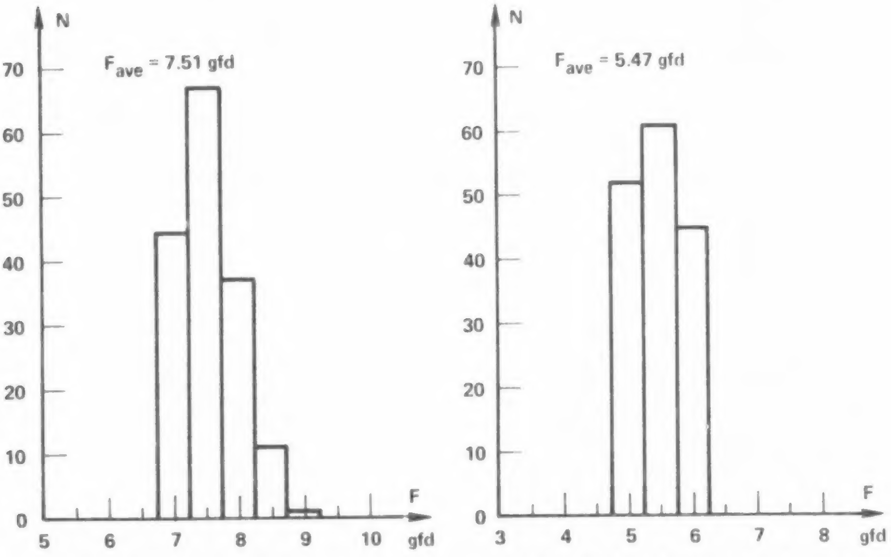


(b) Computed Quantities

Figure 5. Typical Overall Reverse Osmosis Unit Performance.



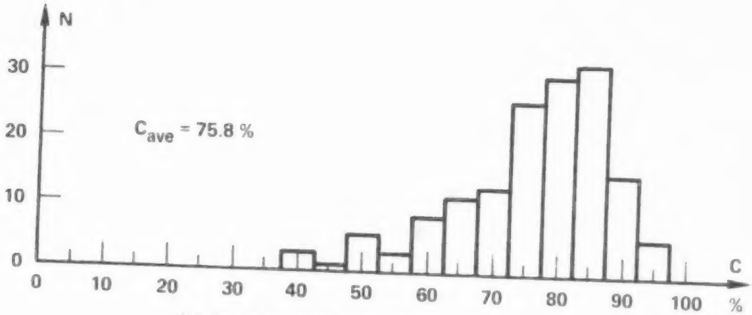
(a) Distribution of values measured after chemical and sponge ball cleaning.



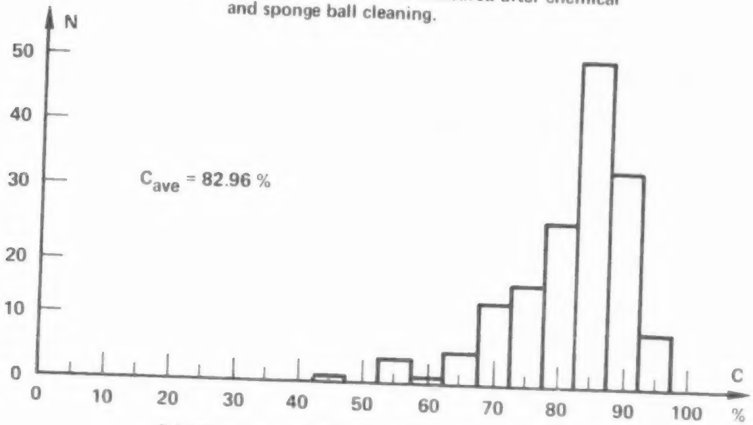
(b) Distribution of values measured the following day.

(c) Distribution of values taken three days after cleaning.

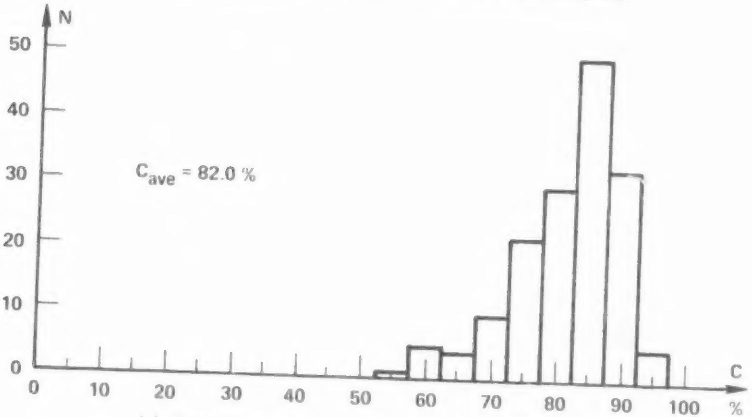
Figure 6. Flux Distribution.



(a) Distribution of values measured after chemical and sponge ball cleaning.



(b) Distribution of values measured the following day.



(c) Distribution of values taken three days after cleaning.

Figure 7. Salt Rejection Distribution.